DEVELOPMENT OF TUNNEL MONITORING SCHEME FOR DIFFERENT GROUND CONDITION

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By

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Declaration

I hereby declare that this thesis entitled "Tunnel Monitoring Schemes for different Ground Condition "was composed by myself, with the guidance of my advisor, that the work contained herein is my own except where explicitly stated otherwise in the text, and that this work has not been submitted, in whole or in part, for any other degree or processional qualification. Parts of this work have been published in [state previous publication]

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Certificate

This is to certify that the Project prepared by Mr. Behailu Getachew Shewaabebe entitled "Tunnel Monitoring Schemes for Different Ground condition" and submitted in fulfillment of the requirements for the Degree of Master of Engineering complies with the regulations of the University and meets the accepted standards with respect to originality and quality

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Acknowledgement

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Abstract

Nowadays underground structures are very important. Based on engineering observations; Properties during geotechnical construction are integral parts of the design of underground structures. The utilization of underground structures, particularly tunnels for storage and trans portion purposes, is a suitable solution for improving life in urban environment, all over the world. Ethiopia is characterized by hilly topography, with large climatic changes throughout the year. Hence, road tunnels are in high demand with respect to protecting traffic from travelling long distance, and in order to lead traffic through the mountains instead of long and winding climbs. Because of these circumstances, there is a need for tunnel construction in different region of the country, Ethiopia. However, Tunnel accident is abrupt to become a serious disaster when it occurs. Therefore, systematic health monitoring is necessary to tunnels during their life time. This paper presents instrumentation as a tool to assist with these measurement observations, determine the need for modifications to loading or support arrangement. Also, apart from above construction control, instrumentation is also indispensable for site investigation, design verification and safety of the structure. Instrumentation used in the construction of tunnels can be implemented in three stages before, during and after construction. Tunnels which are constructed in populated area and have a more comprehensive instrumentation and monitoring program that additionally includes monitoring of ground conditions, underground water levels, tilt and settlement of nearby buildings or other structures of interest in the vicinity of the tunnel alignment. Instrumentation monitoring for tunnels includes monitoring of the structures under Construction together with the ground, buildings and other facilities within the predicted zone of influence. Furthermore, instrumentation and subway tunnels in and around them increase accuracy of the different layers of the earth and excavation of the surrounding structures and make safety and accuracy. This paper presents the features of sophisticated instrumentation available today for geotechnical monitoring. A wide variety of these instrumentation have been described with their applications and also different equipment schemes used to meet the requirement of different types of structure for different ground condition.

CONTENTS

List of tables and figures

Key Words and Their Definition

Design update	Update of the tunnel design based on additional information regarding the ground properties and system behavior
Expected system behavior	Range of system behavior identified during tunnel design, considering the respective scatter of the ground properties. It fulfils the requirements of ultimate limit state, serviceability limit state and project specific requirements
Ground behavior	Reaction of the ground to the excavation of the full profile without the consideration of sequential excavation and support
Ground condition	Condition of the ground in terms of composition, stiffness, strength, structure, hydrology, etc.
Ground displacements	Displacements of soil or rock surrounding an underground Structure
Ground type	Ground with similar properties
Mitigation measures	Measures in order to reduce the probability of an unfavorable event, e.g. increase of support, supplementary measures, etc.
Monitoring	Measurement of physical parameters
Monitoring data	Values obtained by measuring certain changes of the system behavior, e.g. displacements, inclinations, strains, water level, pressure, etc.
Monitoring program	Compilation of means and methods for monitoring the system behavior
Observation	Active acquisition of qualitative and quantitative information
Observational method	Continuous review of the behavior and update of the design and adjustment of construction method

	during construction, based on actual conditions And observations, as required. Tender design is based on the assessment of the most probable conditions and likely variations
Observation variables	Variables with special relevance to the system behavior
Resolution	The fineness of detail that can be distinguished in a measurement
Surface settlements	Vertical displacements of the ground surface
System accuracy	Resulting accuracy from instrumentation and evaluation
System behavior	Behavior resulting from the interaction between ground, excavation, and support System stability of both ground and support
NATM	New Austrian Tunnelling Method
ADECO-RS	Analysis of Controlled Deformations in Rock and Soils
GG	Gilgel Gibe

CHAPTER ONE

1.INTRODUCTION

1.2General

Tunnel construction nowadays requires adequate monitoring for detecting both any signs of instability as well as preventing environmental impacts on tunnel's vicinity(nearby). Monitoring has always been a part of tunnel design, whether it is simple visual assessment of conditions or by using sophisticated automatic electronic device. Based on stability and strength of the subway tunnel and by instrument stability is a measure of confidence in the estimates. The initial cost of the used instruments, always look for a way to get the most information with the least utility. Deformation measurement instrument is installed in the tunnel roof and at the selected walls to monitor tunnel roof and at the selected points along the tunnel wall to monitor vertical, horizontal and longitudinal stress deformation components. The number of points and their detailed location depends on the size of tunnel and excavation sequencing in multiple drift applications. And also, a minimum of for each drift including temporary elements should be equipped with the device capable of measuring deformations. The designer of geotechnical construction works with naturally occurring materials and does not find their exact engineering properties. He may carry out tests in the laboratory on the samples picked up from the field, and sometimes change the naturally occurring materials to make them more suitable for his needs. Instrumentation is used to measure the response (deformation, stress and etc.) of soil or rock to changes in loading or support arrangements, and from the measurements taken, the need for modifications to the loading or support arrangements are determined. This illustrates the basic reason why instrumentation is generally, of immense value during geotechnical construction. Instrument data helps the Engineer to determine how fast construction can proceed without adverse effects on the foundation soil and construction materials used. Safety of instruments can provide early warning of impending failure. In case of metro railway tunnels instruments provide early warning through real time monitoring systems for any excessive and underground movements affecting the adjoining premises, structure and utilities like the railways, power lines, waterlines etc. In this side, legal protection based on instruments provide designers and contractors the basis of a legal defense should resident and owners of adjacent properties blame of construction for damage to their property and life. This aspect gains prominence in constructions in populated areas such as for underground metro railways. Example, monitoring leakage, pore water pressure and deformation can provide an indication of the performance of a tunnel. Monitoring loads on rock bolts and movements within a tunnel can provide an indication of the stability of tunnel

1.2 Objective

General objective: -

To develop tunnel monitoring scheme for different ground condition

Specific objective: -

Monitoring of displacement and stress around a tunnel underconstruction.
Monitor the integrity of the tunnel lining

1.3 Methodology

Monitoring and measuring is one of main factors of Tunnel excavation method in different ground condition. By measuring the tunnel rock deformation data, collecting and analyzing it, measuring data can be used for the feedback of excavation results and for guiding the construction timely. The tunnel monitoring includes monitoring methods, monitoring equipment, data processing analysis, and through data analysis and feedback to guide the construction. Tunnel monitoring and measurement is carried out at a given tunnel site in the present day. By processing and evaluating the vault settlement and surrounding displacement measurement data of the tunnel section, the deformation characteristics and development trend of the tunnel rock is analyzed. Thus, the design and construction of the tunnel is amended and optimized. The stability of the tunnel is forecast and ensured. The result of monitoring and measuring in the tunnel was effect. And it avoided big accident caused by the landslides or the large deformation of the supporting structure due to excessive deformation of the surrounding rock. Referring different literatures from internet in tunnel instrumentation and monitoring system. From this literature review we gather different type of tunnel instrumentation and monitoring program. Therefore, I identify the typical hazards which is happened due to the tunnel construction, how can be observe this hazard and what kind of instrumentation is important to monitor the hazard. Accordingly monitoring program includes the specification of the measuring procedure, the location of the monitoring devices and the monitoring schedule. Attention shall be paid to the fact that the monitoring results are often affected by instrumentation, Installation and environmental effects. Finally, we can prepare a well-organized tunnel monitoring scheme for different ground condition

Chapter Two

2. Literature Review

2.1 General

Tunnel monitoring is an important task in civil engineering that aims at determining the stability and safety of a structure by using information about its deformations. In this paper we present the development and the results of a fast method for displacement measurement based on digital images, which allows a deformation analysis along the cross-sections of a tunnel. This technique was created to overcome some drawbacks of most traditional sensors, especially in the case of underground lines, when time becomes an issue of primary importance and limits the number of cross-sections that it is possible to check. The method was tested inside a laboratory and inside a real gallery, by using several digital cameras and different target configurations in order to determine the best compromise in terms of precision and survey time. The accuracy was checked by giving some displacements with a micrometric sledge capable of moving reference target along prefixed directions. In addition, a control over a period of some months was carried out to test the stability of the method under different conditions in terms of temperature, humidity and illumination. The experimental results demonstrated an accuracy better than ±1 mm for a tunnel wider than 12 meters, notwithstanding the shorter data acquisition time and the low cost of the cameras used. [1] Structural health monitoring is a growing field of research that is attracting increasing interest from government agencies in order to maintain the safety of buildings, dams, tunnels and civil infrastructures. For each of these structures there are different monitoring approaches and technologies. Some real applications are presented in Brownjohn (2007). In several real surveys only, a combination of different instruments can provide sufficient information to monitor the deformations and predict the behavior of the structures. Thus, several sensors are today used, featuring different characteristics in terms of accuracy, cost, Time needed for their set up and so on. However, in the case of complex structures only a monitoring system based on combined sensors allows a complete and detailed analysis of the object and its surroundings, with different accuracies according to the requests of civil engineers. In addition, multiple measurements taken with different sensors can be used to check the consistency between dissimilar technologies. The state of the art about systems that combine multiple sensors can be found in Hill and Sippel (2002). A real application is presented in Alba et al. (2010). Nowadays, deformation monitoring with Conventional Terrestrial surveying instruments can be achieved via intensive manual observations or with automatic sensors, which are permanently fixed on the structure. This choice depends on many factors: the period of the monitoring, the level of risk for people or infrastructures potentially involved, the shape of the object, the measurement accuracy, the installation costs, the number of investigated points and their distribution and so on. For these reasons there are many applications where manual measurements are steadily used, for example when operational infrastructures are subject to temporary works that actively or passively have an effect on the stability of the structure (e.g. the excavation of an underground line). [1] It is essential to detect cracks and exfoliations of tunnel concrete linings at an early stage in view of safety and durability of tunnels. At present, visual inspections and percussion tests have been normally carried out for the purpose. However, these inspection techniques have a large artificial error, and are restricted depending on the timing and locations, such as restrictions in highway constructions and in scheduling of train headways. We have continued our studies at the premises of Railway Technical Research Institute on an inspection method with optical fibers and electric conductible paint as a technique of monitoring damages occurred in tunnel concrete linings. Asa damage detection technique with optical fibers, we adopted the method, which enables to locate the positions and the sizes of strain from dispersion of a light as called the Brillouin dispersion. Where any crack occurred in concrete lining to which electric conductible paint previously applied, the paint is fractured at the location of crack that result in stoppage of electric current. Taking advantage of the result, the said method enabled to detect crack. We were able to verify the function of defect detection with manufacturing of a model of tunnel lining and specimens, and with optical fibers, electric conductible paint and carrying out loading tests. Furthermore, we proved that there was no problem in durability or practical usage by carrying out measurements inside an existing railway tunnel for a long time. This research disclosed a defect surveillance technique with optical fibers and electric conductible paint was satisfactorily effective [2]. It is important to detect any change and cracks generated on tunnel concrete lining at an early stage, in terms of tunnel maintenance. The general surveillance method of tunnel concrete lining is the visual check and the survey of the displacement which carried out by manpower with survey apparatus. However, they are not adequate for constant surveillance of a mile-long tunnel Then, we developed a method to monitor automatically the strain and the progress of cracks with installation of an optical fiber sensor in tunnel concrete lining. Moreover, we also developed another method to detect cracks automatically by checking the resistance value of the electric conductible paint applied on the tunnel lining. We report the contents of the development regarding these two surveillance methods. [3]

2.2 Tunnel Monitoring System Using Optical Fiber Sensor

We regarded the optical fiber sensor as a method applicable to continuous remote surveillance of the damage data: the grade and positions of damages, the progress of cracks and exfoliation of tunnel lining. Then, we made developments for their adaptability of the optical fibers to tunnel lining and its installation method. [3]

2.3 Summary on Optical Fiber Sensor

Although there are various aspects in the optical fiber sensor, we adopted Brillouin Optical Time Domain Reflection (BDTDR) method. With the method, it is possible to detect the position of cracks generated and the size of strain covering a length of more than 10 kilometers by dispersion of a light called the Brillion dispersion. The error in this strain measurement is 100 micrper meter. [4]

2.4 Loading Experiment for a Tunnel-Lining Model

We performed loading experiments to verify the adaptability to tunnel lining deformations with a tunnel model of 1/3 size to investigate the difference in the measurement result of adhesion method, optical fibers were adhered on the model by three different adhesion methods. In addition, the optical

fiber was adhered in the direction of the circumference of the tunnel model We applied an experimental vertical load on the crown of tunnel lining [4].

2.5 Field Experiments

The 1/3 model experiment showed that crack and strain were detectable by using an optical fiber with sufficient accuracy. However, an actual railroad tunnel has inferior environment, such as strong wind pressure by train running, damage on tunnel lining produced by surrounding rock pressure, seepage from a crack, etc. Therefore, we verified the durability and construction characteristic during the experiment in an existing tunnel. Furthermore, we carried out a field experiment at an inclined shaft bottom of the Shinkansen tunnel to assume the tunnel, which receives strong wind pressure by passing trains. We proved that the optical fiber sensor under the BOTDR method had sufficient applicability to damage surveillance of tunnel linings based on the model experiments and the field experiments. However, the cost required to install the foregoing devices in this technique is normally expensive. As continuous monitoring takes place at many sections for a long time, it become advantageous from a conventional technique such as measurement of tunnel interior diameters, measurement with a strain gauge instrument, measurement with instruments to detect cracks and others. Moreover, it is an optimal technique when a change of deformation behavior of the entire tunnels must be broadly investigated. [4]

2.6 Detecting Method for Tunnel Lining Cracks Using Electric Conductible Paint

The authors have studied the system, which adopted electric conductible paint by making the crack of tunnel lining into an economical detectable method. This system has the following features:

(1) Detection of simple and obvious cracks is possible.

(2) Detection is possible under any circumstance.

(3) The system is very economical as a new damage supervising method of tunnel, an optical fiber sensor incorporated under the BOTDR method and a crack detection system with electric conductible paint. This research proved that both of techniques were suitable for damage surveillance of tunnels. While technical developments actively progressed for maintenance management of civil engineering structures including railway tunnels, it is conceivable that these techniques are promising as a new damage surveillance technology. Optical fibers are highly likely incorporated in disaster prevention systems, which cover various ground structures including tunnels. From now on, we increase records of accomplishment of these techniques to the field and believe that we are able to improve the damage supervising system with higher practicality. [3]

CHAPTER- 3

3.1. HYDROPOWER TUNNELS IN ETHIOPIA

3.1.1 Introduction

Tunneling is new in Ethiopia and has been recently used in hydropower projects. Till now, tunnels have been built in six hydropower projects such as Fincha hydropower, Beles hydropower, Tekeze hydropower, and the three Gilgel Gibe hydropower projects. In this chapter, the Gilgel Gibe I, II, III and Beles hydropower tunnels will be reviewed briefly. The construction method used in the GG I and GG III is drill and blast whereas TBM excavation method used in the excavation of GG II and Beles hydropower tunnels.

3.2 Gilgel Gibe I Hydropower Tunnels

Gibe I is located at about 260 km south-west of Addis Ababa on the road to Jimma town. Gibe I headrace tunnel was constructed by Conventional drilling and blasting method and its length is about 9300m. The Powerhouse was constructed underground, in a 68,000m3 Cavern, 98m long,38m high and 18m wide, excavated inside the massive limestone rock on the right bank of the river [5].The construction methods, techniques and procedures of the GG I hydropower tunnels are the same to that of the GG III hydropower tunnels.

3.3 Gilgel Gibe II Hydropower Tunnels

a) Location

The Gibe II is located at 240 km south-west of Addis Ababa, coordinates (7° 45′ 25″ N,37° 33′ 44″ E). The project channels the water discharged from the Gilgel Gibe I dam through a26km long tunnel and a steep drop directly to the valley of the Omo River [6].

b) Design

The Gilgel Gibe II consists of a power station on the Omo River that is fed with water from ahead race tunnel and sluice gate on the Gilgel Gibe River. The headrace tunnel runs 26 km under the Fofa Mountain and its internal diameter is 6.98m, and at its end, it converts into a penstockwith a 500 m hydraulic head/drop. The design discharge was 100m3/s and the design internal pressure was minimum 2 bar and maximum 7 bar. As shown in Figure 3.1, the tunnel has a diameter of 6.98 meters and extends for 26 kilometers and is designed to withstand a maximum pressure of 7 bar (0.7 MPa). The tunnel is buried deep in the ridge formed by Fofa Mountain at depths varying from 300m to 1.3km at its deepest section [6]. The ridge through which the tunnel is dug is made up of a non-uniform rock formation with several types of rocks with the dominant ones being basalt on the west side followed by trachyte and rhyolite on the east side of the tunnel. As the figure shows, the ridge is intersected by numerous faults generally inclined in the east direction [5].

I. Main summary of ground Investigations

The rock mass rating (RMR) was between 17 and 19. The rock class V - very poor. No groundwater was encountered but the rock was only damp. The temperature was varying from 42ºc to 53ºc. Five rock formations along the tunnel alignment. Mainly tertiary volcanic rocks: rhyolite, trachyte, basalt: Uniaxial compressive strength of intact rock (σc) = 120 Mpa Number of joints per meter = 6Orientation of

discontinuities with respect to tunnel axis = Oblique Stand up time >48 hrs. Ground water inflow at tunnel face = 0 liters /sec [5]. discontinuities with respect to tunnel axis = Oblique Stand up time>48 hrs.

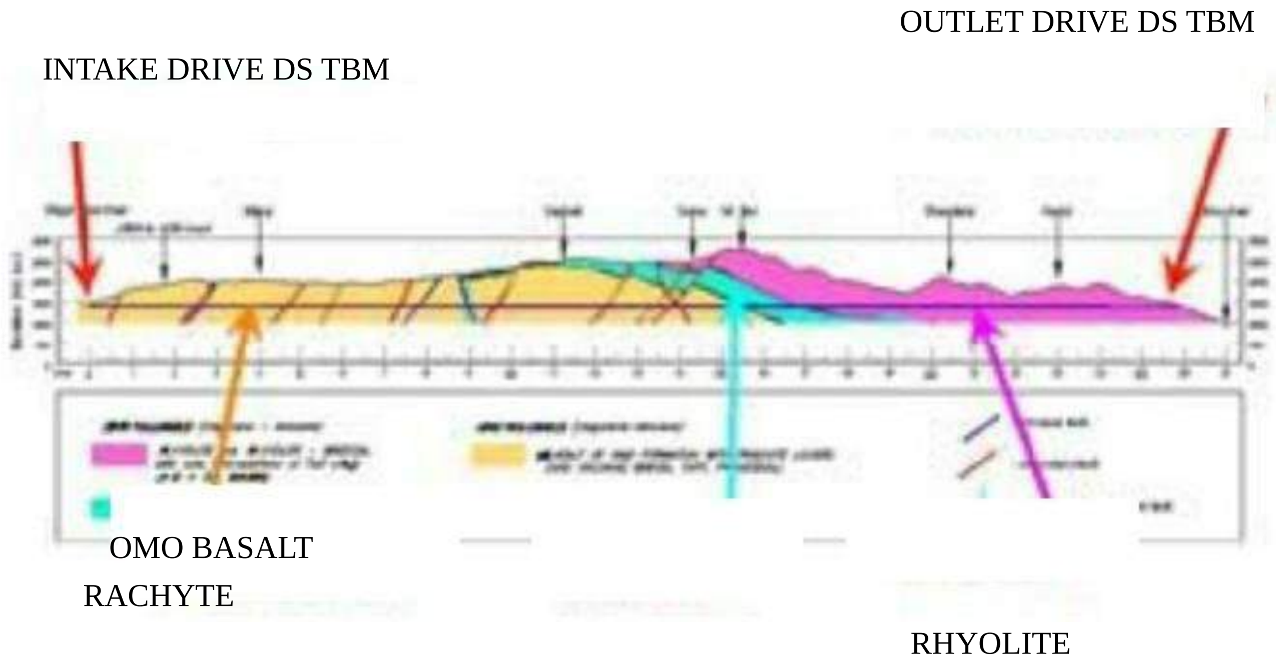


Figure 1. Gilgel Gibe II geological profile [7]

ii. Geometrical Characteristics of Lining [5]

Segment Type ……………………………………………. Hexagonal 4 pieces
Segment Thickness ……..……………………………….25 cm
Segmental Lining Outside Diameter ……….………..... 6,800 mm
Segmental Lining final internal diameter…………………6,300 mm
Segment Width …………………………….…………. 1,600 mm.

c) Challenges and Remedial Measures of the collapsed section

The first failure happened, during the construction of the tunnel that was started in June 2005.InOctober 2006 the tunneling crew hit a pocket of wet earth along a major fault line only 4 heading. Wet and humid mud under a pressure of as much as 40 Bars and with temperatures reaching 40ºC gushed out. This immense pressure which is almost 6 times the maximum pressure of 7 Bars that the tunnel was designed for not only damaged the tunnel itself and its linings but also the 255-ton TBM. A summary of the failures as reported by the tunneling engineering consultant read as follows: ‘Event at the chainage 4+196 from intake heading. At the end of October 2006, the TBM was pushed back as consequence of the sudden extrusion and collapse of the tunnel face against the cutter head and the front shield. The tunnel face moved towards the TBM 40-60 mm/hour. The TBM has been pushed back more than 60 cm and displaced laterally more than 40 cm. As consequence, severe damages occurred to the shields, the cylinders and the last 7 segment rings installed behind the TBM. ‟ [8]The second failure occurred in June 2007 resulting in the collapse of the front face of an exploratory adit (access tunnel) which eventually filled 80m of the main tunnel with mud. Further, in August 2007, unexpected fault line was crossed during excavation; but with no reported damage at that time [8].

I. Remedial Measures: The solutions to these series of unforeseen events involved.

1. Building a new 230meters of bypass tunnel,
2. Changing the original layout (direction) of the tunnel,
3. Dredging as much as 40,000 m3 of mud continuously for 2 years (between October 2006August 2008),
4. Drilling of 1,600m of drainage/exploratory holes [5], and
5. Filling out the failed section of the tunnel with concrete. On December2007, monitoring measurements have shown lowering of the rock stresses, while the right exploratory adit was crossing the fault zone [5]

ii. Further Remedial Measures

Completion of the back chamber through a concave shot-concrete wall, reinforced with horizontal steel ribs HEB200; Construction of a new assembly chamber and TBM launching chamber; Segmental lining dismantling and casting of a concrete plug in the power tunnel; Resuming of the intake drive excavation along a new alignment at Chainage 3+805.To facilitate TBM steering at the excavation resuming and to reinforce the pillar between the two tunnels, in the transition zone the old tunnel has been filled with concrete. The TBM has been refurbished and re-assembled in the tunnel [5]. The excavation diameter was enlarged from 6980mm to 7074mm peripheral cutter housings (from n°37 ton°44) were repositioned. These remedial measures, constituted significant departure from the original design. [7]. The success of the intervention was possible only after the releasing of the pressure and the stresses acting in the area and surrounding the TBM. Large mud/water inflows, unstable faces of raveling/running and blocky ground, high rapid convergences, high ground loads, and very hot water and gas inflows were conditions faced by the crews of two double shield tunnel boring machines (DS TBM) working towards each other from the portals. After starting in October 2006, the machines met in June 2009 and the tunnel was watered-up in September 2009 [9].

iii. Tunnel Collapse after Inauguration and repair in January 2010, about 8,500m3 of material, ranging from silt and mud to blocks of rock up to 2-3m wide, crashed through a 15m long section of the 8.10m diameter TBM driven segmentally lined tunnel, bringing operation of the facility to a standstill just three months after inauguration [20]. About 1.3 km of the tunnel was inaccessible because of the rocks that blocked passage and trapped the water. It was also stressed that a large quantity of rock and mud fall into the tunnel [8].

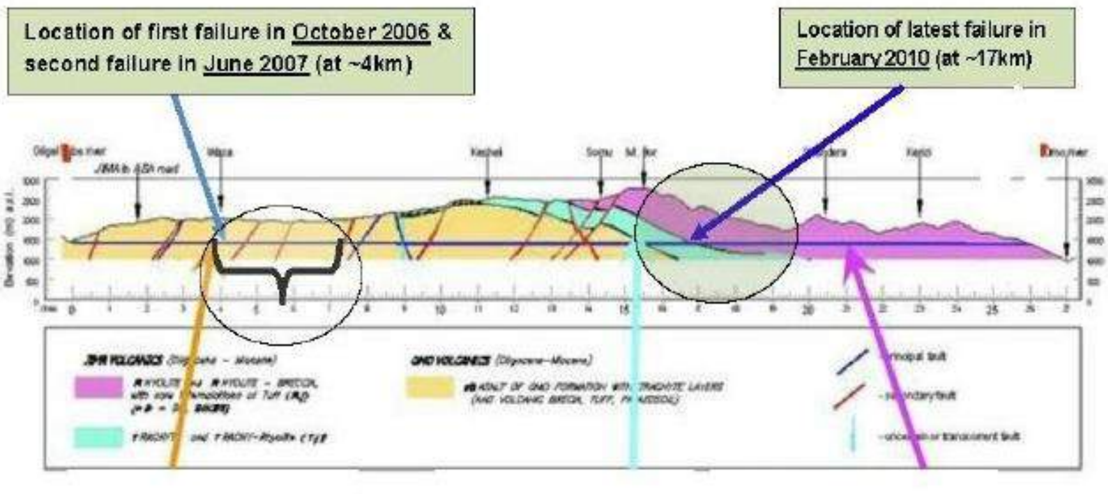


Figure .2 Cross-section of the tunnel showing multitudes of fault. Lines Later failures showed that there were still more fault lines unforeseen at the beginning of the project [8].

3.4Evaluation of Construction Methods and Excavation Techniques

Tunnels can be constructed by using a number of construction methods and techniques (TBM, roadheader and drill and blast). Although there is a difference between soft ground and hardground (rock), tunneling techniques are being used in a wider range of ground conditions and this boundary is becoming increasingly blurred. It should be noted that from a practical point of view, if the ground is stable, the construction of the tunnel can focus on economics and be driven by the limits of the tunneling equipment. For soft ground, which needs immediate support, construction is driven by the need to support the ground immediately after creation of the void [10].

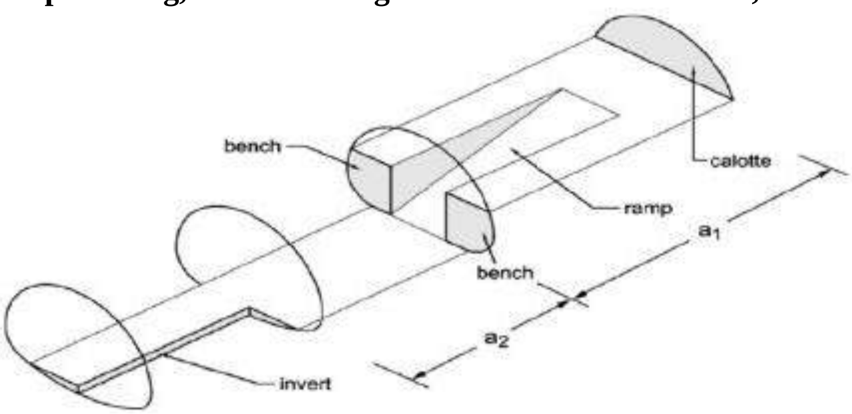
3.4.1. Excavation Method

The GG III hydropower tunnels consist of about twelve tunnels, namely, power tunnels, river diversion tunnels, drain and access tunnels. The ground of the GG III hydropower tunnels is stable and its rock properties are classified into three classes, class II, III and IV. The GG III hydropower tunnels are relatively short and are not straight. The length of each tunnel is less than 2km. The diameters of the GG III hydropower tunnels are variable due to the rock conditions (see Appendix F). The shape of these tunnels, except the left river diversion tunnel, is horse shoe. For these conditions, therefore, drill and blast excavation method, used in GG III HPTs, is more economical rather than TBM or road header excavation methods. [10]

3.4.2. Excavation Techniques

Large cavities are less stable than small ones. Therefore, in many cases the tunnel cross section is not excavated at once, but in parts called partial face excavation. According to NATM, the most widespread methods of partial face excavation are (i) top heading and bench, and (ii) sidewall drift. Top heading and bench is a type of excavation techniques in which the crown is excavated first and then the bench is excavated (Figure 4.4). A soon construction of a temporary invert of the crown section or, better, the soon excavation and support of the bench and invert helps avoiding large settlements of the abutments of the crown arch. This means that the length a = a1+ a2 (Figure 4.5) should be kept as small as possible. On the other hand, a1 should be sufficiently large (1 to 1.5 times the tunnel diameter, depend on the rock classes/properties) to enable efficient excavation and support works in the crown [11]. The top heading and bench excavation technique is implemented improperly in the construction of the GG III HPTs.

Figure 3. Top heading, cross and longitudinal sections. 1: calotte, 2: bench [11]



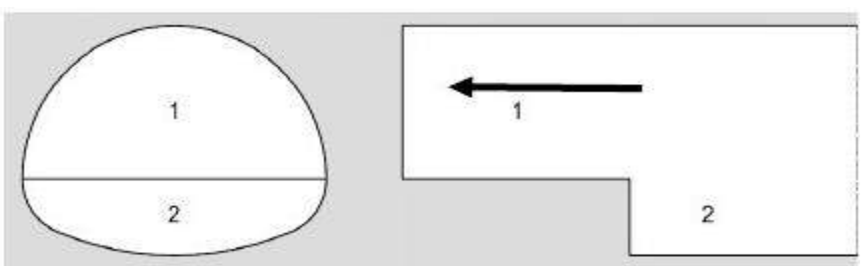


Figure. 4 Schematic representation of top heading [11]

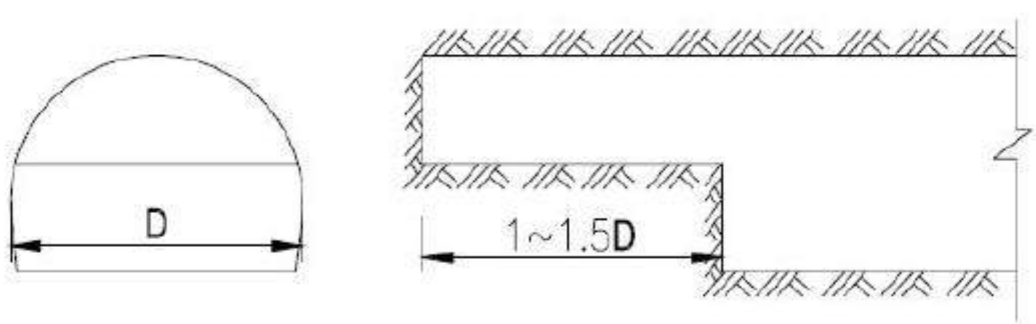


Figure 5. Sketch of two-step (top heading and bench) excavation technique [12]

In GG III hydropower tunnels, Portals of all tunnels excavated in full face excavation technique whereas the full length of the tunnels excavated in top heading and bench excavation technique. But the top heading excavation technique was not implemented in the proper way, i.e. The bench excavation has been started after completion of top heading excavation of the full length. The control of convergence and settlement of the abutments and crown of the top heading is principally achieved by providing early invert closure [10]. In the construction of the GG III HPTs, no provision of early invert closure was made. The progress of bench excavation after completion of top heading excavation the control of convergence and settlement is also principally achieved by limiting the length of advance per stage [10]. But in GG III HPTs, the length of advance in the bench excavation was 15m to 20m per stage. This is a huge blast in a tunnel construction and its disturbing factor is very high in the crown arch which has been excavated in top heading [13]. The excavation technique used in the GG III HPTs will lead to large settlements and convergences of the abutments and crown. This settlements and convergences also lead to wedge (rock and shotcrete) failures and finally tunnel section collapse. Challenges have been faced in construction of the GG HPTs due to this excavation technique and the support requirements. The author has observed over break tunnel cross-sections in the power tunnels (Figure 6 and Figure 7). These over break tunnel x-sections required to be filled with concrete mortar. The cast-in-place reinforced concrete permanent tunnel lining was designed to be 65cm and 70cm in thickness however, due to these over break voids, around the permanent tunnel linings, the quantity of concrete required to fill the voids is absolutely very large as shown in the figure below figure 6

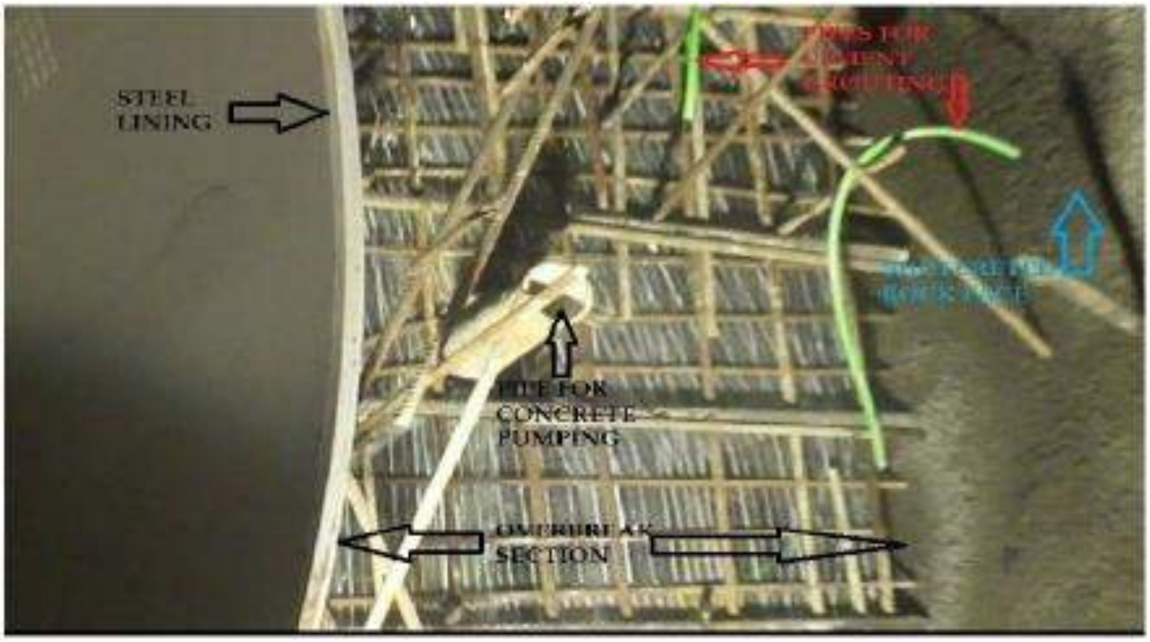


Figure 6. Typical over break sections and steel lining in the power tunnels

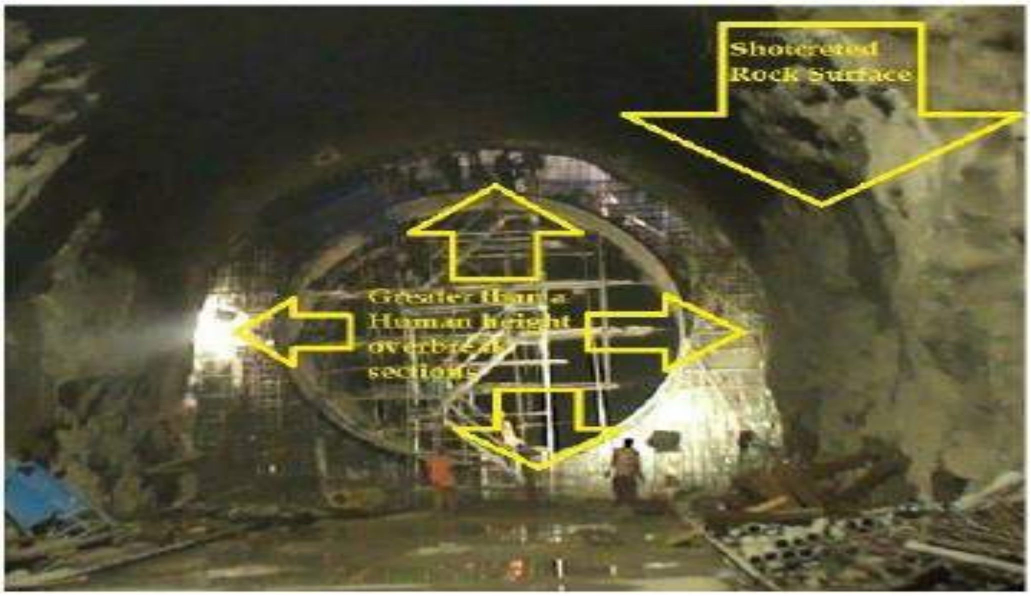


Figure 7. Typical over break sections and steel lining in the power tunnels This is a lack of controlled blasting knowledge and it, therefore, leads to uncontrolled cost overrun.

3.4.3. Lining/Support

Sprayed concrete (shotcrete) lining uses an incremental excavation sequence and sprayed concrete used as a primary (initial) support with, or without, weld mesh, fibers, lattice arches girders), dowels, anchors and bolts. The primary support details are to be determined in advance of the construction by the designer and are to be validated during construction by instrumentation and monitoring devices [10, 11, 14]. prayed concrete lining (SCL) or shotcrete, welded wire mesh, rock bolting and cement grouting are used in the GG III hydropower tunnels as some initial support systems whereas cast-in-place reinforced concrete and steel linings used as a final lining together with cement grouting. These lining systems are suitable for the drill and blast construction method. As discussed above, the excavations executed with over break tunnel cross-sections. The designer accomplished clearly the length of drilled holes and the length of blasted rock portions together with charging and blasting patterns for the two-phase excavation techniques. The cast-in-place reinforced concrete finial lining was designed to be the thickness of 70cm and 65cm.However, due to lack of controlled blasting problems, the final lining thicknesses obviously increased. Thus, the volume of the required concrete and grouting also increased incredibly because the over break voids are to be filled with concrete and then cement grouting. [14]

3.4.4. Challenges Encountered in Gilgel Gibe III

Certain challenges occurred in the GG III hydropower tunnels during its construction. Especially, a major collapse occurred in the central river diversion tunnel, and rock and shotcrete wedges failure in the excavation of the central diversion and right power tunnels as discussed below. [9]

3.4.5. Challenges with regard to geotechnical parameters

Challenges were not recorded with regard to the geotechnical parameters, i.e., no changes in geological structures, rock properties, ground water conditions, etc.

3.4.6. Challenges with regard to construction Procedures and support requirements

Rock wedge failures occurred in power and diversion tunnels during tunnel construction at different locations. A major rock wedge failure occurred in the right power tunnel at manifold section at chainage 0+843. A man died due to this rock wedge failure when a shotcreting of the manifold section is on progress (Figure 4.12 and Figure 4.13). The rock class is II, RMR = 69. This type of rock might have remained stable for about six months for only 8m of unsupported excavated span, according to the RMR method. In the manifold section, the power tunnel branched in to five tunnels each of which will take water in to five turbines. Therefore, the excavated width, span and height, in the manifold section, are more than 8m.

Figure 8. Rock Failure on the Shotcreting Machine [25]

Figure 9. Rock Failure Section at chainage 0+843[13]

The rock wedges failure encountered in the central river diversion tunnel at stations 0+175,0+178 and 0+185 during excavation. A major collapse occurred after 74 days of excavation at chainage 0+175 and this collapse extended into both directions (backward to station 0+160 and forward to station 0+250). The causes of this collapse and remedial measures that are used to repair the collapsed section will be discussed below. The investigations of causes were carried out by Pietrangeli (Designer, from Italy) and Lombardi Engineering (from Switzerland). They used the recorded geological reports (for example bore holes of A10 and E1) (Figures 4.14), recorded actual face mappings (Table 4.3), visiting the collapsed section, etc. The recorded in-place face mappings and rock classifications carried out during excavations of the collapsed area are listed in Table 4.3. The rock wedge failure occurred at station 0+178 during excavation but the section collapsed at station 0+175 after 74 days and 67m advance of the construction. This collapse extended into both directions, therefore,these data are used for cause investigations.as shown on the table 1

Table 1

	Mapping date	RMR	Rock class	Remark
Chainage0+148	18-Feb-2007	57	III	Fair rock
0+164	21-Feb-2007	59	III	Fair rock
0+169	23-Feb-2007	55	III	Fair rock
0+178	01-Mar-2007	39	IV	Poor rock*
0+204	18-Mar-2007	57	III	Fair rock
0+218.5	27-Mar-2007	57	III	Fair rock
0+228	14-Apr-2007	57	III	Fair rock
0+242	28-Apr-2007	57	III	Fair rock

	Mapping date	RMR	Rock class	Remark
Chainage0+148	18-Feb-2007	57	III	Fair rock
0+164	21-Feb-2007	59	III	Fair rock
0+169	23-Feb-2007	55	III	Fair rock
0+178	01-Mar-2007	39	IV	Poor rock*
0+204	18-Mar-2007	57	III	Fair rock
0+218.5	27-Mar-2007	57	III	Fair rock
0+228	14-Apr-2007	57	III	Fair rock
0+242	28-Apr-2007	57	III	Fair rock

Table 1. As built rock classifications of the collapsed area [15]

Installation of inappropriate ground excavation initial supports. The Lombardi Engineering Limited remedial works report [26] said “all along the tunnel, the rock surface was covered with skin shotcrete and some pitons (swellex)” As explained above, the collapsed section is rock class IV (RMR = 39). The initial supports that were designed for rock class IV include:

Shotcrete Thickness ≥25cm,

Welded Wire Mesh: ϕ5mm, 10cmx10cm,

Pattern Rock Bolt: 23ϕ25mm, L= 6m@1.5m with cement grouting

These supports were not installed in place. As said above, the installed supports were skin (very thin layer) shotcrete and some (spot) swellex. This indicates that the quality control body has not carried out his obligation. Drainage holes were not installed. The Lombardi Engineering Limited remedial works report [15] said “during the excavation no water was reported, however some dripping and wet shotcrete observed around the collapsed area at the moment of the inspection. It is assumed that the water reduces the cohesion of the filling of the discontinuities increasing the loads on the supports. The installed supports were not able to resist the new loads and some blocks collapsed and then other blocks lost their lateral support, finally, the collapsed sections enlarged rapidly into both directions”, from chainage 0+175 backward to 0+160 and forward to 0+250. [15].

3.4.7CONCLUSIONS AND RECOMMENDATIONS

The objectives of this research were to evaluate the design and construction approaches, to assess challenges encountered in the design and construction of the GG III hydropower tunnels, and to recommend future tunnel design and construction considerations based on the lessons learned from the GG III hydropower tunnels. Conclusions and recommendations are presented below as well as suggestions for future study also have been proposed. Recommendations for future Tunnels in Ethiopia the author recommends the following based on the problems encountered in the design and construction of the GG III hydropower tunnels.

1. The most globally used empirical design methods such as the geomechanics rock classification (RMR) and the rock mass quality (Q) shall be used in the Design and construction of tunnels.
2. In the empirical design methods, all the design parameters shall be evaluated/used during design and construction processes.
3. In the GG III HPTs, Rock science software products, e.g. phase 2, unedge, etc.… are used for the design analysis. But these software's mainly used the GSI values as input. Since the correlation values of the GSI with RMR are inappropriate for the RMR values greater than fifteen (15), it's better to use discrete element methods (DEM) or finite element methods(FEM).
4. Whatever the contract type, the consulting company shall carry out his obligation Properly during design and construction processes.
5. There must be installation of controlling instrumentations to control convergence and settlement in a tunnel construction.
6. Tunnel construction is very challenging and full of risks. Thus, technically skilled and experienced persons shall participate in the tunnel design and construction Processes because the geological and geotechnical uncertainties can be tackled effectively using proper rock classifications on site. Finally, the author suggests for any researcher, he/she can do his/her investigations on the following: -

1. Verification analysis for the relationship of GSI with RMR using different modelling software.
2. The reasons of Gigel Gibe II headrace tunnel collapse and the remedial measure

CHAPTER FOUR

4.1 NECESSITY OF GEOTECHNICAL INSTRUMENTATION

The designer of geotechnical construction works with naturally occurring materials and does not know their exact engineering properties. He may carry out tests in the laboratory on the samples picked up from the field, and sometimes change the naturally occurring materials to make them more suitable for his needs. But his structural design will essentially be based on engineering the values of properties of the materials tested by him. Therefore, as construction progresses and exact geotechnical conditions observed or behavior monitored by means of instrumentation, the design judgments are evaluated and changes made, if necessary. Hence observations by means of monitoring instruments during geotechnical construction are an integral part of the design process. Instrumentation is a tool to assist with these observations. They are our eyes and ears inside the rock. Instrumentation is used to measure the response (deformation, stress etc.) of soil or rock to changes in loading or support arrangements, and from the measurements taken, the need for modifications to the loading or support arrangements is determined. This illustrates the basic reason why instrumentation is generally of immense value during geotechnical construction.

4.2Purpose of good instrumentation

A good instrumentation program should have one or more of the following purposes in mind:

4.2.1 Site investigation

Instruments are used to characterize and determine initial site conditions. Common parameters of interest in a site investigation are pore pressure, permeability of soil etc.

4.2.2 Design verification

Instruments are used to verify design assumptions. Instrumentation data from the initial stage of a project may show the need or provide the opportunity to modify the design in later stages. For example, data obtained from NATM shotcrete cells in the initial stretch of tunnel is used to revise the thickness of shotcrete in the later stages.

4.2.3 Construction control

Instruments are installed to monitor the effects of construction. Instrument data helps the engineer to determine how fast construction can proceed without adverse effects on the foundation soil and construction materials used. For example, in tunnel construction, the data obtained from the load cells helps the geotechnical engineer to know if the stresses in the excavated tunnel have been stabilized and how fast he can proceed with further excavation.

4.2.4 Safety

Instruments can provide early warning of impending failure. In case of metro railway tunnels instruments provide early warning through real time monitoring systems available on the internet for any excessive and undue (beyond the normal control) ground movements affecting the adjoining premises, structure and utilities like the railways, power lines, water lines etc. within the zone of influence of the excavations or tunnels. This allows for implementation of preventive remedial actions well within time.

4.2.5 Legal protection

Instruments provide designers and contractors the basis of a legal defense should resident and owners of adjacent properties blame construction for damage to them property and life. This aspect gains prominence in constructions in populated areas such as for underground metro railways.

4.2.6 Performance

Instruments are used to monitor the in-service performance of a structure. For example, monitoring leakage, pore water pressure and deformation can provide an indication of the performance of a tunnel. Monitoring loads on rock bolts and movements within a tunnel can provide an indication of the stability of tunnel. [5] The primary purpose of geotechnical instrumentation is to monitor the performance of the underground construction process in order to avoid or mitigate problems. The primary function of most instrumentation is to monitor performance of the construction process in order to avoid or mitigate problems. There are, of course, other related purposes, and proper management of the program will include decisions on which of the following deserve primary consideration and which may be considered of lesser importance:

- To prevent or minimize damage to existing structures and the structure under construction by providing data to determine the source and magnitude of ground movements.
- To assess the safety of all works by comparing the observed response of ground and structures with the predicted response and allowable deformations of disturbance levels.
- To develop protective and preventive measures for existing and new structures.
- To select appropriate remedial measures where required.
- To evaluate critical design assumptions where significant uncertainty exists.
- To determine adequacy of the Contractor’s methods, procedures and equipment.
- To monitor the effectiveness of protective, remedial and mitigative measures.
- To assess the Contractor’s performance, Contractor-initiated design changes, change orders, changed conditions and disputes.
- To provide feedback to the Contractor on its performance.
- To provide documentation for assessing damages sustained to adjacent structures allegedly resulting from ground deformations and other construction related activities.
- To advance the state of the art by providing performance data to help improve

future designs.

CHAPTER FIVE

5. INSTRUMENTATION AND MONITORING OF TUNNELS

5.1 Introduction

The primary purpose of geotechnical and structural instrumentation is to monitor the performance of the underground construction process in order to avoid or mitigate problems. If such monitoring also serves a scientific function, or leads to advancement in design procedures, that is a bonus rather than a primary reason for its implementation. Afew decades ago monitoring was not a particularly easy task because the tools were few and some not so well developed. Monitoring was generally performed manually, and the refining of data to a state of usability from the raw readings often required long hours of "number crunching" with relatively crude calculators and more long hours of plotting charts and graphs by hand [17]. The world of the early 21st century is very different for those who pursue the art of determining what ongoing construction is doing to its surroundings, or even to itself. Advanced and refined types of instrumentation abound, and electronics coupled with computers have made remote monitoring, even from half a world away, practically an everyday affair. It is common for even medium sized projects to run a computerized database that reduces raw readings to usable data and can report on any combination of instruments and data plots within minutes. It can also inform interested parties any timeof the day or night if movements or stresses have reached pre-set trigger levels that demand some kind of mitigate action. The possibilities have not gone unnoticed by project Owners, and comprehensive instrumentation and monitoring programs are becoming the norm rather than the exception. This is perhaps especially true in the world of tunneling where even small miss-steps can result in damage that may lead to lawsuits or the shutting down of operations. [16]

5.2 Tunnel Deformation

5.2.1 Purpose of Monitoring

When the temporary or permanent structural support for a tunnel is being designed, calculations are performed to predict the kinds of movements and stresses the support can safely be subjected to before there is danger of failure. It is the job of instrumentation specialists to track those movements and stresses and provide guidance on whether the support or the construction process needs to be modified to ensure short term safety and long-term stability of the completed tunnel. For braced excavations it is standard practice to measure the loads on some of the support members, and often to combine these with measurements of the support member deflections if the measurements of ground movements outside the support system are not sufficient to present a complete picture of support performance. It is possible to thus monitor the significant performance related behavior of soldier piles, slurry walls, struts, tiebacks and other elements of open cut or cut-and-cover excavations. In mined tunnels it is generally more common to use deflection measurements as a first line of defense against adverse developments because the eccentricities in the movements of many support members, such as steel ribs, make stress and load measurements much more complicated and prone to varying interpretation than they are for braced excavations. [18]

5.2.2 Ground deformation measurements

The objectives of ground deformation monitoring are different in mountain and urban tunnels. In mountain tunnels, the main objective of deformation measurements during construction is to ensure that ground pressures are adequately controlled, i.e., there exists an adequate margin of safety against collapse, including roof collapse, bottom heave, failure of the excavation face, yielding of the support system, etc. Adequate control of ground pressures ensures a safe and economical structure, well adapted to the inherent heterogeneity of ground conditions. This procedure is compatible with modern tunnel design methods which include a range of excavation and support systems to cover the anticipated spectrum of conditions along the tunnel, with selection of the applicable system in each case relying on the encountered geology at the tunnel face, experience on tunnel behavior at previously excavated sections under similar conditions and, on accurate deformation measurements, i.e., by applying the so-called "observational method". This method of construction can ensure adequate safety and, at the same time, an economical construction. On the contrary, in urban tunnels, the main objective of ground deformation monitoring is to limit ground displacements to values sufficiently low to prevent damage to structures and utilities at ground surface. Thus, the fundamental difference in deformation monitoring stems from the fact that in mountain tunnels the objective is to guard against an ultimate limit state (i.e., collapse) while in urban tunnels the objective is to guard against serviceability limit states (i.e., crack initiation) for structures and utilities at ground surface. As a result of these differences in objectives,
design philosophies, and construction techniques, the types and required accuracy of the measured ground deformations vary between the two classes of tunnels, as follows:

In mountain tunnels: -

Considerable ground deformations are deliberately permitted (and often provoked) in order to reduce the initially very large "geostatic" loads on the temporary support by increasing ground de-confinement. Such reduction of ground loads on the tunnel support can be appreciable and, thus, extremely beneficial provided that excessive "loosening" of the rock mass is prevented (such "loosening" can cause roof failures and an eventual increase of the ground loads). De-confinement is achieved by controlled inward ground deformation at the excavation face (face-take), controlled delay in the completion of the temporary support measures (by increasing the distance from the face where the tunnel invert is closed), a relatively flexible temporary support system (e.g. long passive rock-bolts and thin sprayed concrete liners) and, finally, by installing the permanent lining at a later time when evolution of the long-term (creep) ground deformations has practically stopped. In extreme cases of strongly squeezing ground conditions, sliding supports maybe installed to permit tunnel wall convergences of several tens of centimeters. In all these cases, control of ground deformations depends strongly on efficient and timely deformation measurements. However, due to the large ground deformations (several centimeters and even several tens of centimeters), the required level of precision of these measurements needs not be excessive; typically, accuracy of the order of one centimeter is sufficient in mountain tunnel applications. [18]

In urban tunnels

The main objective is limiting ground deformations around the tunnel and thus causing the minimum possible movement and disturbance at ground surface and the structures founded there.

This is achieved by: -

- ✓ Limiting inward ground deformation at the excavation face (face-take), e.g. by facepre-reinforcement using fiber-glass nails, stiff steel beams (fore-poles), cement-or jet-grouting techniques,
- ✓ By installing a stiff temporary lining, usually including invert closure, as early as possible
- ✓ By installing the final lining as quickly as possible, especially when tunnel wall convergences continue to evolve with time.

The above “stiff” construction methods tend to reduce ground de-confinement and thus the ground loads on the tunnel lining are a significant fraction of the initial “geostatic” loads. Such loads often are not a problem, since urban tunnels are usually shallow and thus the initial” geostatic” loads are much smaller than those in deep mountain tunnels. Due to the small ground deformations induced by tunneling (usually less than10mm aground surface and occasionally less than 5mm), measurement precision and the early installation of the measuring devices is of utmost importance. [18]

5.3 Monitoring Project

5.3.1 Workflow

During a geotechnical monitoring project, all items and courses of action, as depicted in Figure1, should be considered. A concept needs to be developed on how to measure the key observation variables of the geotechnical problem under consideration. This includes selection of instrumentation type, monitoring layout, frequency of readings, da transmission and the associated evaluation method. The different work stages, as presented in Figure 10

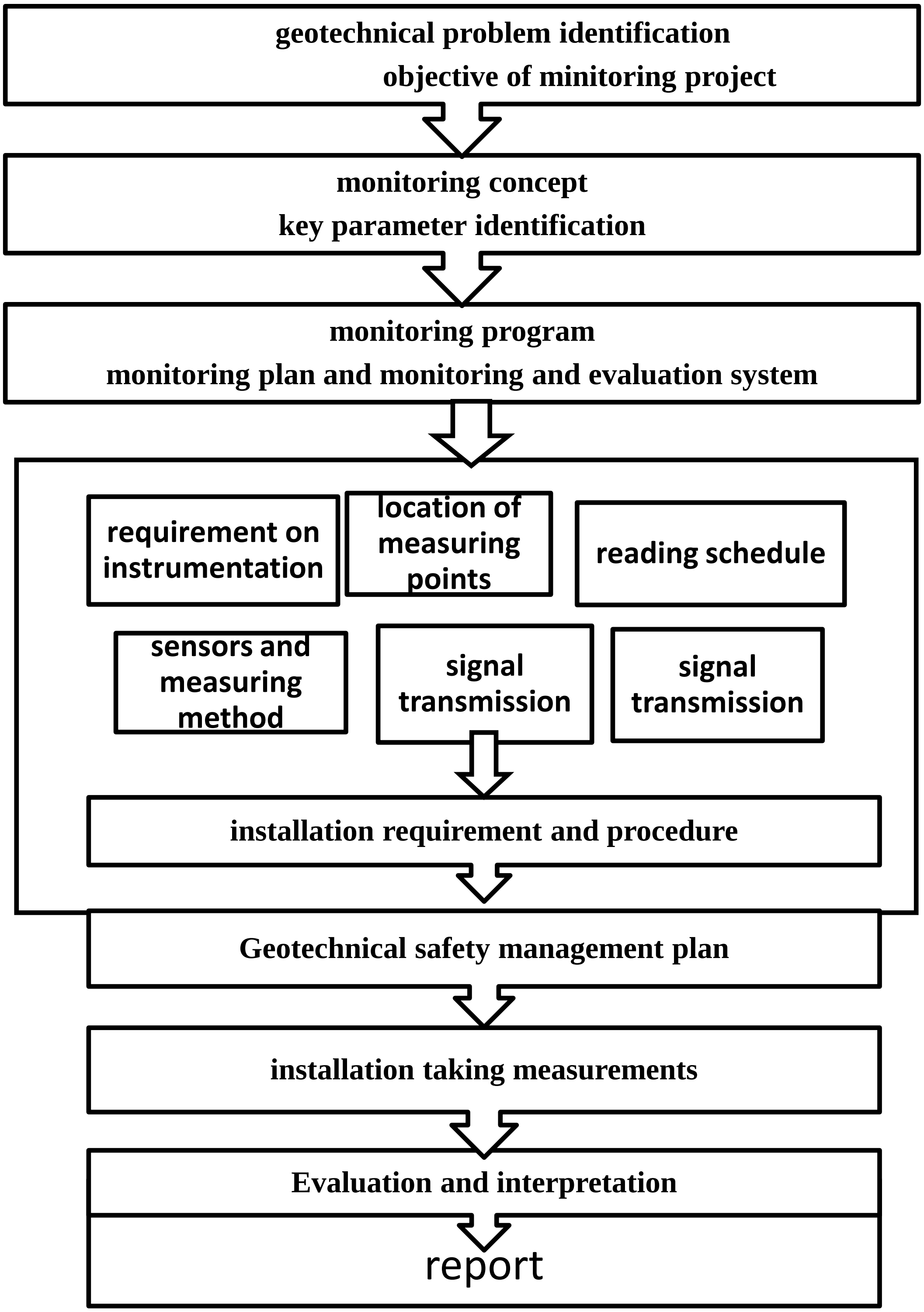


Figure 10. Basic concept of a geotechnical monitoring project. [19]

5.3.2 Definition of expected system behavior

The designer defines the expected system behavior resulting from tunneling, and its impacts on the surface, before construction commences. The following information must be available:

- ➢ Range of expected tunnel displacements for all construction phases
- ➢ Level of loading of the shotcrete lining in all construction phases
- ➢ Surface settlements and their spatial distribution
- ➢ Ground deformation in the area of structures
- ➢ Allowable distortion or curvature of installations and structures in the expected influence area of the underground construction, e.g. buildings, railway tracks, water pipes, wastewater sewers, gas pipes

5.4 Monitoring program

The monitoring program includes the specification of the measuring procedure, the location of the monitoring devices and the monitoring schedule. Attention shall be paid to the fact that the monitoring results are often affected by instrumentation, installation and environmental effects. The chosen type of instrumentation shall guarantee the following:

- ❖ Feasible installation procedure
- ❖ Durability over the monitoring period
- ❖ Protection against damage during construction
- ❖ Simple measurement handling (data acquisition and transmission)
- ❖ Accuracy as required

5.4.1Typical monitoring layouts

In this section typical monitoring layouts for various ground and boundary conditions are shown for illustration. Typically, 3D displacement measurement (in the tunnel and if required on the surface), extensometers, inclinometers, strain gauges, piezometers, tiltmeters, hose water levels, and invert probes are used to observe the system behavior in underground structures. Additional instrumentation and tools, such as compass-clinometer or digital face mapping techniques may be required.

5.4.1.1Typical distances between monitoring sections

Monitoring sections in tunnels and shafts are typically situated at distances of 5 – 20m depending on the boundary conditions and requirements.

5.4.1.2 Monitoring for different ground conditions

Case 1: Shallow tunnel in soft ground beneath groundwater table

Figure 11 Typical hazards: surface settlements; drawdown of ground water table leading to consolidation, instability of face and unsupported ground, day lighting collapse, support failure

Focus observation on: -

1. Deformation of surface, buildings, and utilities
2. Water table drawdown
3. Face stability
4. Lining displacements and integrity

Instrumentation: -

a. Absolute displacement monitoring, levelling, tilt meters, hose water level, extensometers, inclinometers

b. Water level gauge, Piezometer
c. Visual inspection
d. Absolute displacement monitoring, face displacement monitoring, invert probe, strain gauges (alternatively and/or supplementary evaluation of the displacement monitoring data), extensometer. [20]

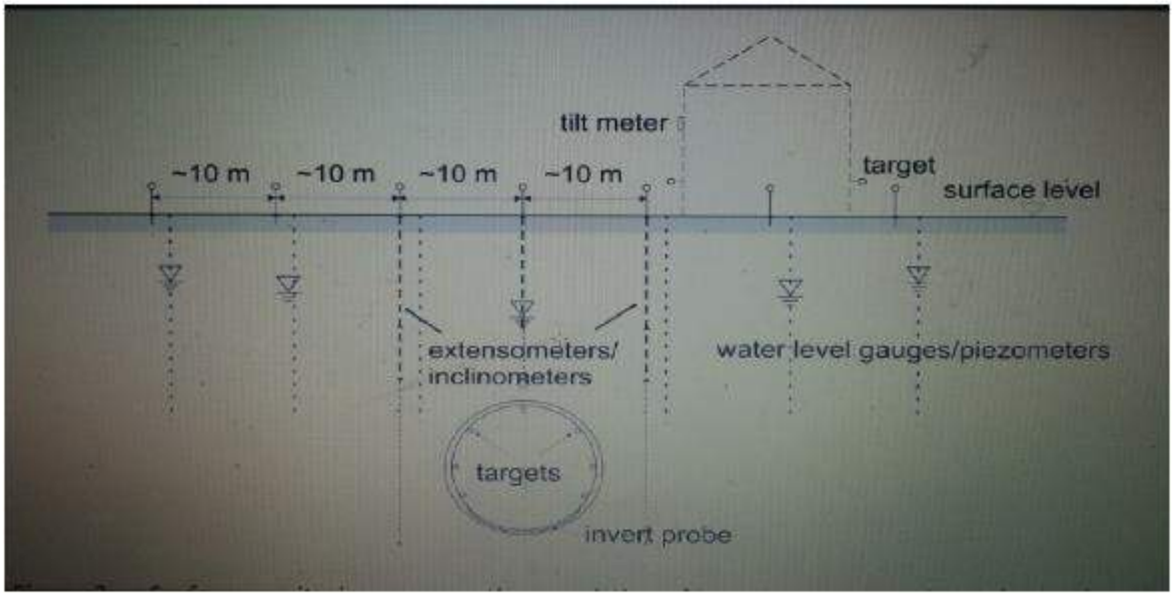


Figure 11: Surface monitoring cross section consisting of targets, extensometers, piezometers, and tiltmeters on the same station of a monitoring section in the tunnel the limitations to the layout on the surface monitoring targets are posed by the morphological conditions and land usage. Typically, the targets are organized in such manner that the construction of settlement profiles (in one line or regular pattern) is possible. In addition, the displacements of buildings are measured with at least three monitoring points, in order to determine both translation and rotation. [19]

Case 2: <u>Tunnel in blocky rock mass - Figure</u> 12

<u>Typical hazards</u>: - Discontinuity induced detachment of blocks

<u>Focus observation on:</u>

- Ground structure
- Discontinuity location and orientation in relation to the tunnel alignment

<u>Instrumentation</u>: - geological compass and/or digital ground mapping Although this type of failure in general cannot be detected in time by displacement monitoring, it is recommended to use supplemental absolute displacement monitoring, albeit in a reduced extent. This guarantees the presence of a fallback monitoring systemin case of encountering adverse ground conditions as shown in figure 12

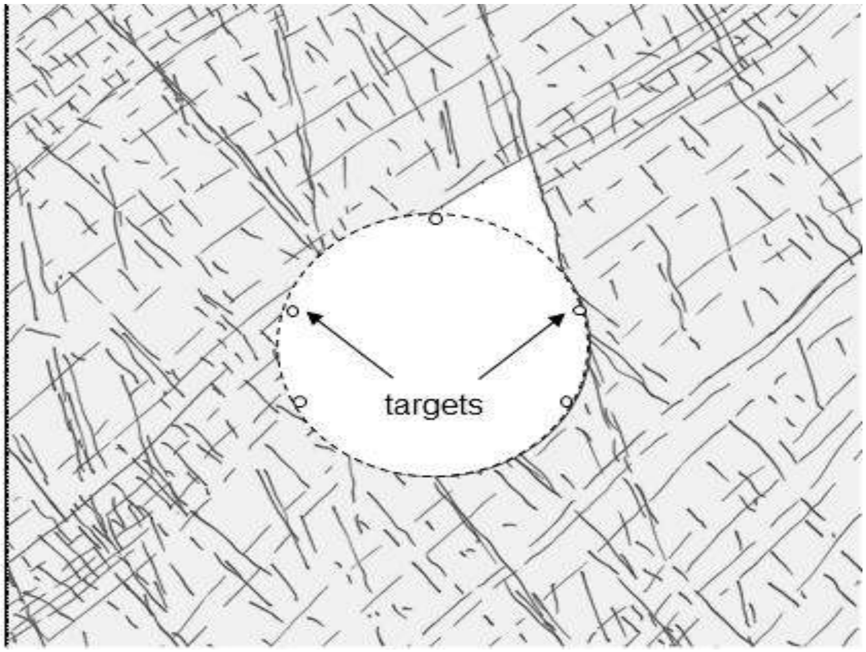


Figure 12. Blocky rock mass with potential for over break; with installed monitoring targets [19]

Case 3: Tunnel with moderate to high overburden in bedded or foliated rock mass

Typical hazards: - violation of clearance profile, lining failure due to shearing along and dilation perpendicular to foliation, large anisotropic deformations

Focus of observation on: -

- Orientation of the bedding or foliation
- Lining displacements
- Ground displacements
- Ground structure

Instrumentation:

1 and 4: compass-clinometer or digital ground mapping

2: absolute displacement monitoring, strain gauges

3: extensometers for identification of mechanically active features

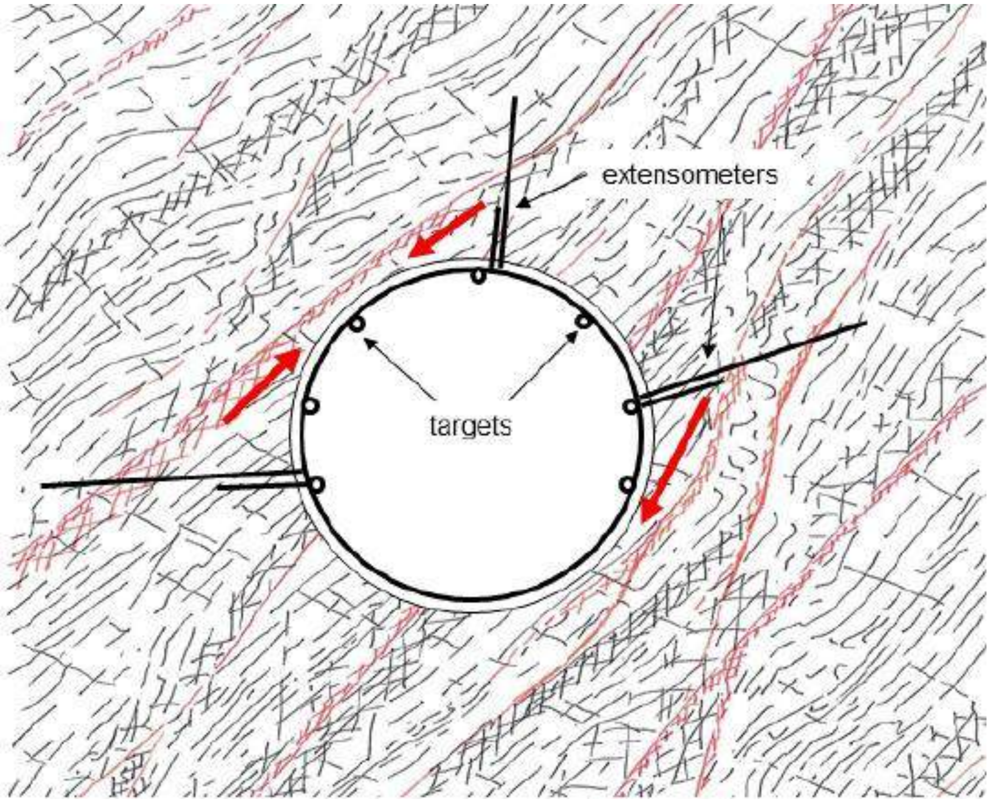


Figure13. Bedded/foliated rock mass with the potential to shear along faults/slickensides/foliation/bedding and dilate perpendicular to foliation/bedding; equipped with monitoringtargets and optional extensometers; arrows indicate assumed shearing along faults [19]

Case 4: Tunnel in swelling ground - Figure 5

Typical hazards: - clearance violation, invert failure due to long lasting increase ofsupport loading

Focus observation on: -

- Presence of water
- Mineralogical composition (swelling clays and minerals)
- Lining condition, deformation and strains
- Ground displacements

<u>observation/instrumentation</u>:

Ad 1: visual inspection

Ad 2: laboratory testing

Ad 3: absolute displacement monitoring, strain meters, invert probe, fiber optical sensors

Ad 4: absolute displacement monitoring, extensometers

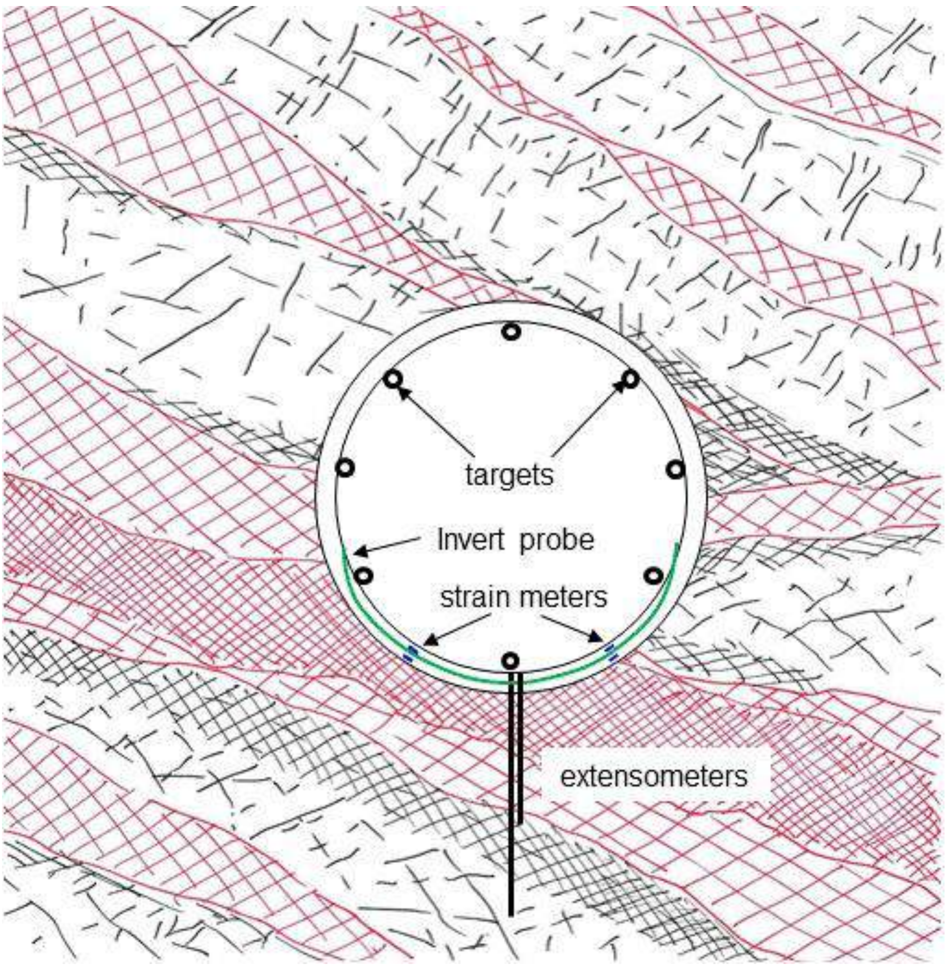


Figure 14. Rock mass with swelling potential; equipped with targets, extensometers combined with measurement point, invert probe, and strain meters installed [20]

5.4.1.3 Monitoring of displacement and stress around a tunnel under construction.

Deformation control during tunneling excavation is becoming over the last decades a basic requirement for both safety purposes and for faster production. The monitoring of deformation may allow to verify if the deformation at the front or in the tunnel are in line with the expected ones. Hence, it is possible to understand if we are excavating under the predicted design condition and, if not, to adopt Effective countermeasures.Italy, together with the Japan has the longest tunnel network in the world, including different types of tunnels and, especially, tunnels excavated in different materials, both hard and Soft rock and soil. Over the last 30 years a new tunneling excavation method has became popular, thus substituting the common NATM method. This method, named ADECO-RS (an Italian acronym for “Analysis of controlled Deformation in rock and soil”) (Lunardi, 2008; Tonon, 2010), considers the deformation as the core of the tunneling activity and, therefore, the control of deformations during the excavation is a key requirement for each planning decision, before and during the excavation phase. Together with the ADECO-RS method, several excavation and stabilization solutions have been developed as well as new approaches for the monitoring of deformations. Hence, deformation monitoring is now at the base of a tunneling project. Furthermore, in case of tunnels crossing an unstable slope several well established traditional monitoring techniques (inclinometers, extensometers, extensor-inclinometers, topographical surveys etc.) are used. Following the great attention given to the observational method, new solutions for the monitoring of slope deformation have been developed in the last years. New branches of deformation monitoring have been created like remote monitoring, i.e. monitoring by lasers, radar etc (Mazzanti, 2012), thus making the monitoring a large and complex science. These techniques are also offering new opportunities such as the deformation monitoring as a tool for investigation purposes (Mazzanti, this volume)..Deformation monitoring surveys were conducted and measurements of movements were carried out throughout the construction cycle of the wall and beyond. In order to determine design parameters for the soil strata embedded in a complex geological sequence, the soil-wall interaction was back analyzed using the finite element method [21].Deformation Monitoring Techniques In this study, currently existing diversified deformation monitoring techniques is being addressed [22].First, Tunnel Wall deformation is usually measured with tape extensometers, geodetic surveying (total stations) and laser scanners (profilometers). Laser scanners (or tunnel profilometers) are recent development in measuring the geometry of tunnel walls in cross- section. A typical system shown in Figure 1 consists of two closed-circuit digital (CCD) cameras mounted on a portable frame [23]. The position of the camera frame is automatically determined by a total station with automatic target recognition placed up to a maximum distance of 100m. For this purpose, three reflector targets are permanently mounted on the frame. Digital images are automatically stored in a computer and can be processed to provide the 3D coordinates of the surveyed tunnel wall surface with an accuracy of F5 mm for each coordinate. Although this level of accuracy is low compared to routine geodetic surveying, the advantage of recording a very large number of points on the tunnel wall can outweighlow accuracy for many applications. Second, deformation measurements at ground surface, structures and utilities are usually performed with surveying instruments (precision levelling for vertical displacement and total stations for 3D geodetic facade monitoring), or with geotechnical [24]

5.5 Monitoring Methods:-

5.5.1 Absolute 3D Displacement Monitoring:-

Over the last two decades 3D displacement monitoring has become common practice,and due to the high information content, has gradually replaced other techniques.

5.5.1.1 Basic principles: -

The measurements are executed by using a total station (tachymeter) and targets.Precise prism-targets as well as bireflex-targets (reflectors) are used and their spatialposition within the global or project coordinate system determined. Discrete threedimensional displacement measurements are executed by repeated measurements(generally on a daily basis). As the complete monitoring generally cannot be carried outfrom one position, an interlinked observation scheme is required, which is establishedusing identical reference points (Figure 15). Stable reference points are differentiated frompoints which are still moving. Points with a defined maximum displacement rate (generally< 1mm/month) can be used as reference points.

5.5.1.2 Requirements

The principle of the “over determined free stationing” is used to obtain the instrumentposition.The absolute position of all coordinate components of marked measuring points shall bedetermined with an accuracy of +/- 1mm (standard deviation) in relation to neighboringmeasuring sections over the whole observation period.To obtain this accuracy, the following conditions should be observed:

- Distance between instrument and closest reference point: 10 - 30 m
- Minimum distance to farthest reference point: 90 m
- Maximum distance to monitoring points: 80 m
- Maximum distance between instrument positions: 110 m
- Use approximately same instrument positions for consecutive readings
- Position instrument on stable ground
- Start free stationing from closest and finish at farthest targets
- Displacement measurement starting from the farthest target
- Make connection sightings approximately symmetrical to tunnel axis
- Intermediate orientation measurements and closing check to selected reference points
- Make zero readings immediately after installation of targets (latest prior to nextexcavation)
- Measuring of all points in one section (in case of zero readings in the bench, topheading targ ets shall be measured as well)
- Recording of meteorological conditions for consideration in the evaluation
- Acclimatization of instrument to avoid deviations due to temperature change

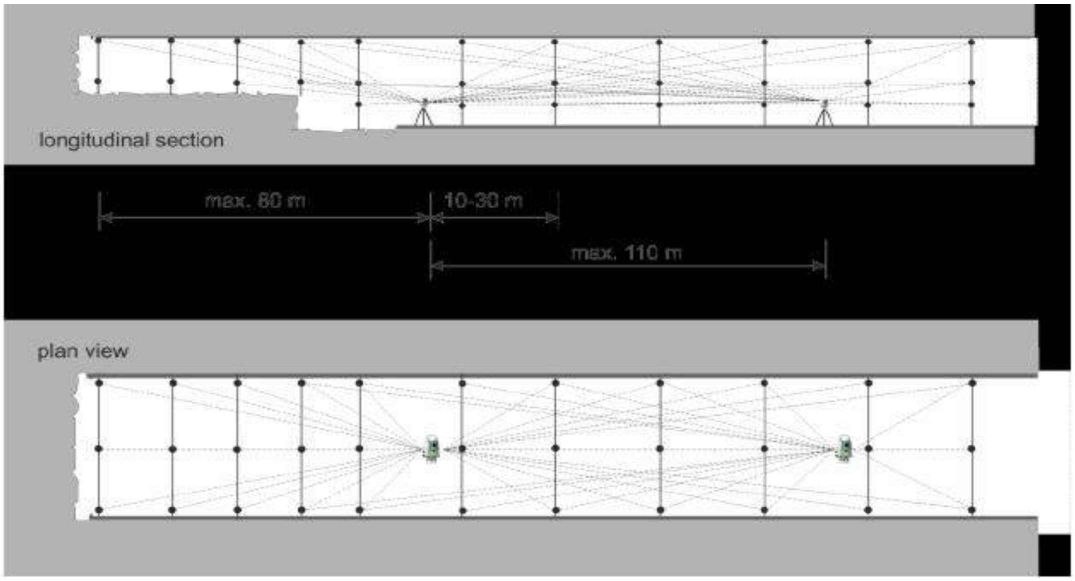


Figure 15. Sketch of an interlinked free station method in the tunnel [25]

The following sources for errors should be avoided:

Sightings close to the tunnel wall (minimum distance sighting to wall 0.5 m to 1 m)

- Measurement errors due to refraction (i.e. sighting through or close to heatsources)
- Instrument position close to sidewall
- Asymmetrical connection sightings (see Figure 16)
- Measurements in very dusty environment or during strong vibrations (i.e. causedby machinery)

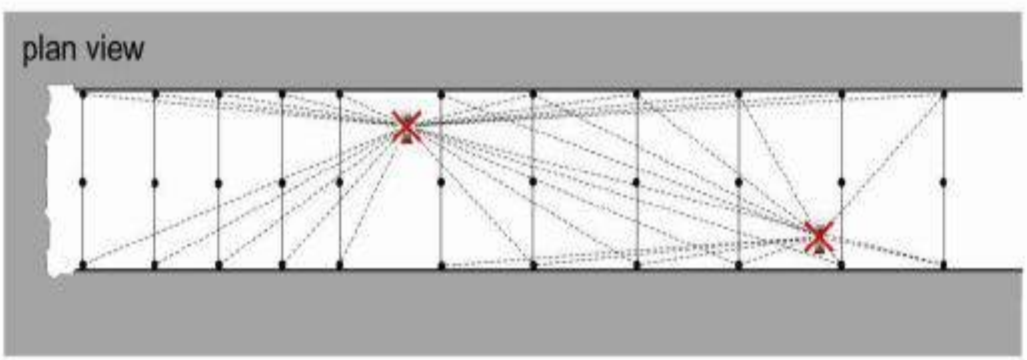


Figure 16. Sketch of asymmetrical connection sightings (to be avoided). [25]

The surveyor shall record and submit the following items after each measuring campaign:

- System of measuring sequence (related to measuring section or along the tunnel)
- Points not measured and indication of reason (destroyed, not visible, etc.)
- Significant displacements (measuring error, rapid increase in displacements)
- Zero readings
- Monitoring conditions (air quality, vibrations, limited visibility, heat sources, etc.)

5.5.1.3 Tachymeter: -

An electronic total station (Figure 17) with automatic recording unit should be used havingfollowing minimum precision:

Horizontal angle measurement: +/- 1’’ (0.3 mgon)

Vertical angle measurement: +/- 1’’ (0.3 mgon)

Distance measurement: +/- (1 mm + 1.5 ppm)

Figure 17. Tachymeter, example Leica, TS 09 .[26]

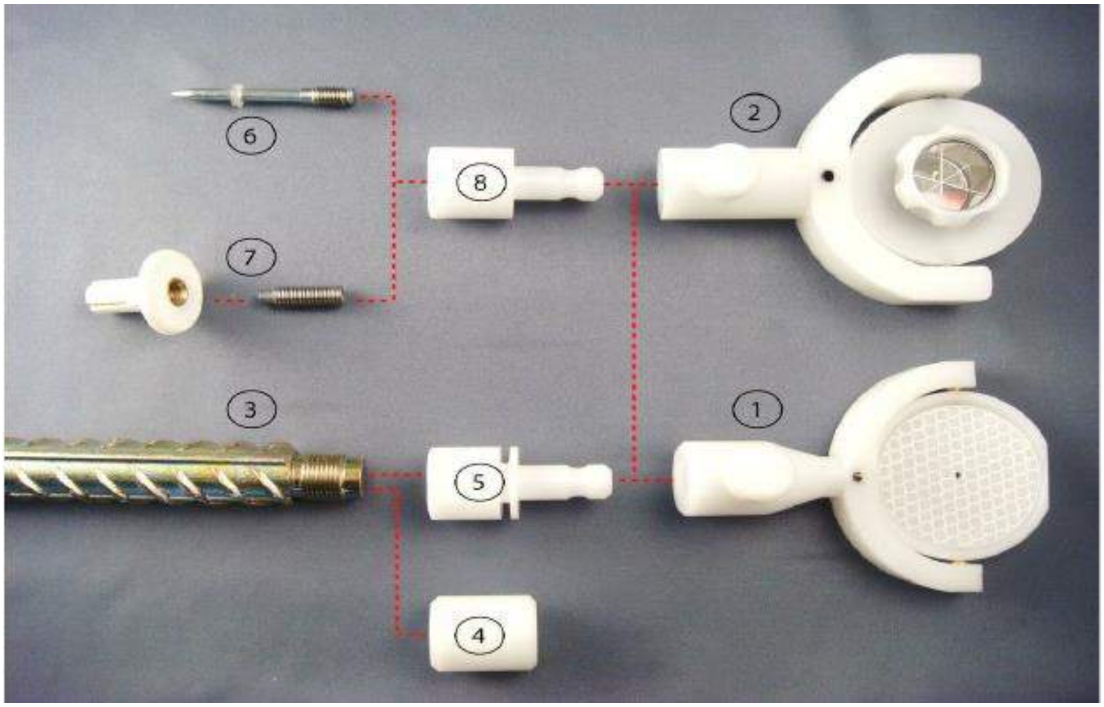


Figure 18. Components of 3D displacement monitoring targets; bi-reflex (1) and prism target (2), bolt (3) with protecting cap (4) and predetermined breaking point (5), mounting bolts (6 + 7) with adapter (8) [2]

The minimum manufacturing precisions required are:

3.5.1.4 Targets

The targets are mounted on special bolts with an adapter (see Figure 18). [26]

Figure 18. Components of 3D displacement monitoring targets; bi-reflex (1) and prismtarget (2), bolt (3) with protecting cap (4) and predetermined breaking point (5),mounting bolts (6 + 7) with adapter (8) [2]

The minimum manufacturing precisions required are:

- Adapter with breaking point: +/- 0.1 mm
- Triple prism target: +/- 0.01 mm
- Bireflex target: +/- 0.1 mm

Required minimum repeatability of readings:

- Triple prism target: +/- 0.1 mm
- Bireflex target: +/- 0.3 mm

This precision shall apply for repeated readings when targets are changed in their orientation or targets/ adapters are replaced

5.1.5 Installation and measurement procedure

To achieve maximum output from the measured data and to ensure comparable measurement results, the following should be observed during underground and surface monitoring:

- A predefined breaking point has to be provided between the bolt and the target, in order to prevent bolt damage
- Solid embedment of the bolt
- Protection against damage during shotcrete installation
- Installed utilities have to be considered when positioning the targets, to guarantee sightings by the tachymeter
- In case of surface targets, the foundations have to be embedded below the frostor permafrost zone
- For surface targets, the target height above ground should be between 50 and 100cm (influence of refraction)

A. Underground

The targets of a monitoring section should be

- Installed immediately behind the face of the last round and the zero-reading taken without delay; deviations from the planned station installation of maximum +/- 1 m are acceptable
- Installed at the same position as those of previous construction phases, i.e. top heading, bench (tolerance +/- 1 m)
- Installed at the same station as the surface monitoring points(tolerance +/- 1m)
- Consistent recording of construction phase (e.g. face positions) and exact assignment to measurements
- The position of the face is determined by the mean value of at least three face position measurements (Figure 10).
- Face position can be determined without target (precision +- 10 cm)
- The new face position is valid as soon as more than 25% of the face Is excavate

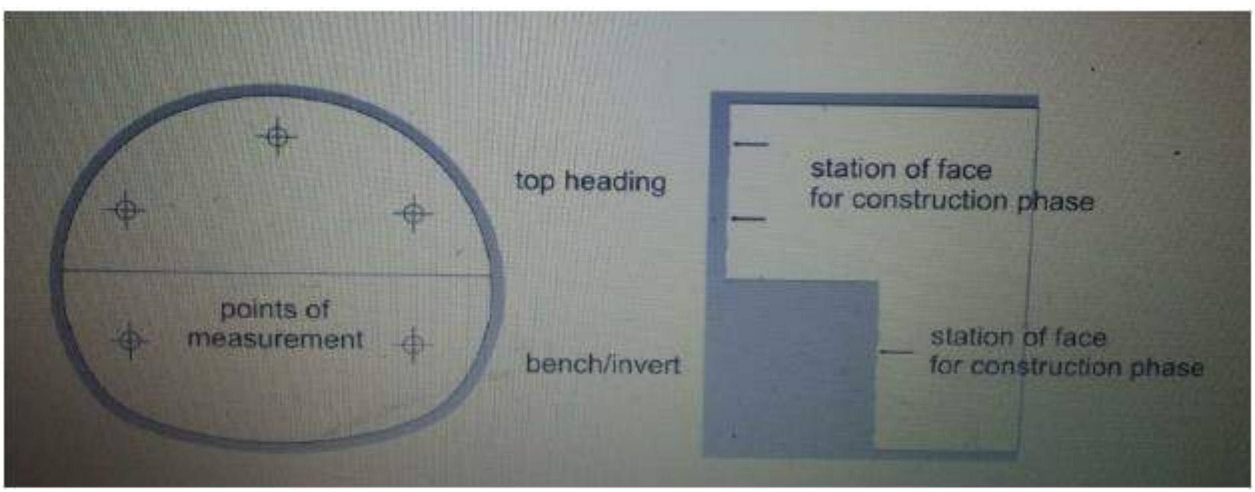


Figure 19. Determination of the face position. [26]

Figure 19 and Figure 20 show the common practice of installation and protection of a monitoring bolt against damage in shotcrete linings. The monitoring bolt and protection cap is fixed to the wire mesh. During shotcreting the bolt should be covered appropriately. The bolt head should be recessed at a minimum 10 mm from the inner surface of the shotcrete lining.

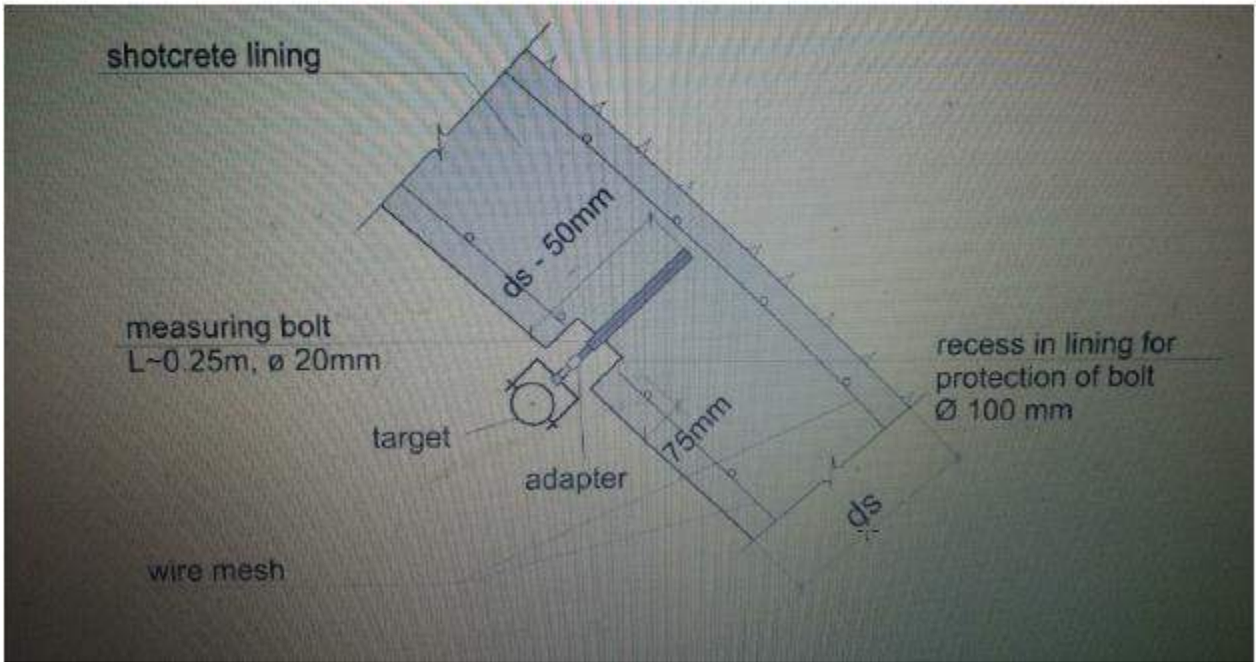


Figure 20. Sketch of monitoring bolt unit mounted in shotcrete lining [27]

Figure 21. Bolt and adapter with prism target mounted in recess [27]

B. Arrangement of monitoring displacement targets

Figures 22 – 26 depict different examples for the arrangement of 3D displacement monitoring targets in tunnels of different size, caverns and shafts. The sketches give an indication of the minimum requirements, which can be adapted in accordance to the local conditions in terms of underground conditions and sensitivity of adjacent infrastructures.

Figure 22. Sketch of monitoring layout for tunnels or galleries with a top heading, bench and invert excavation sequence [27]

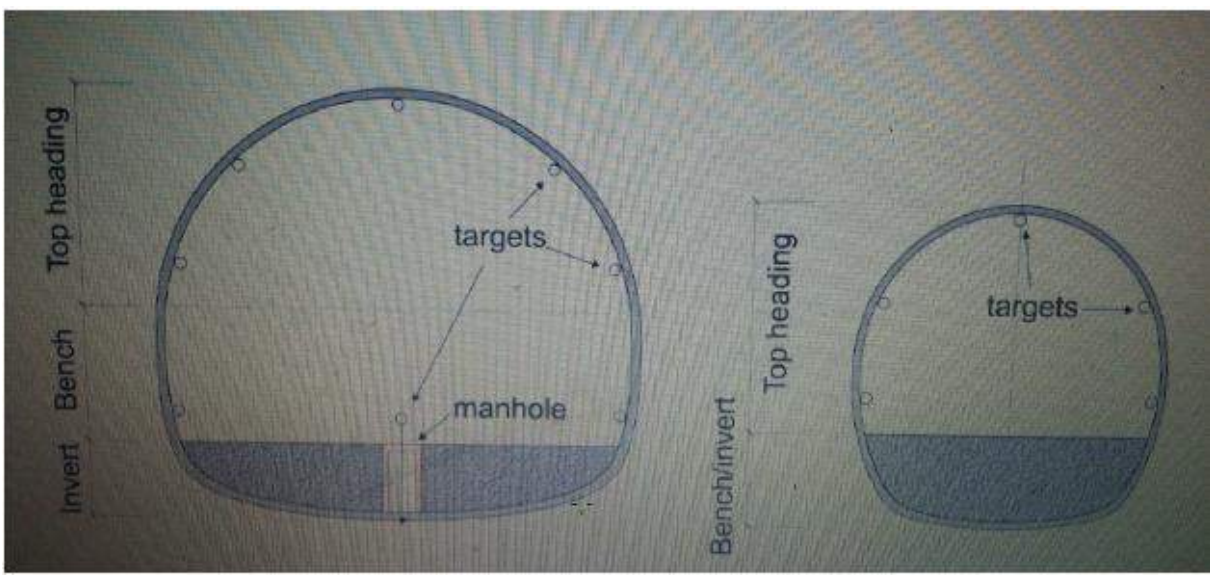


Figure 23. Sketch of monitoring layout for cavern or tunnel with sidewall galleries. [27]

Figure 24. Arrangement of 3D displacement monitoring targets in a tunnel with side wallgalleries during construction as designed in Figure 15 (right). [27]

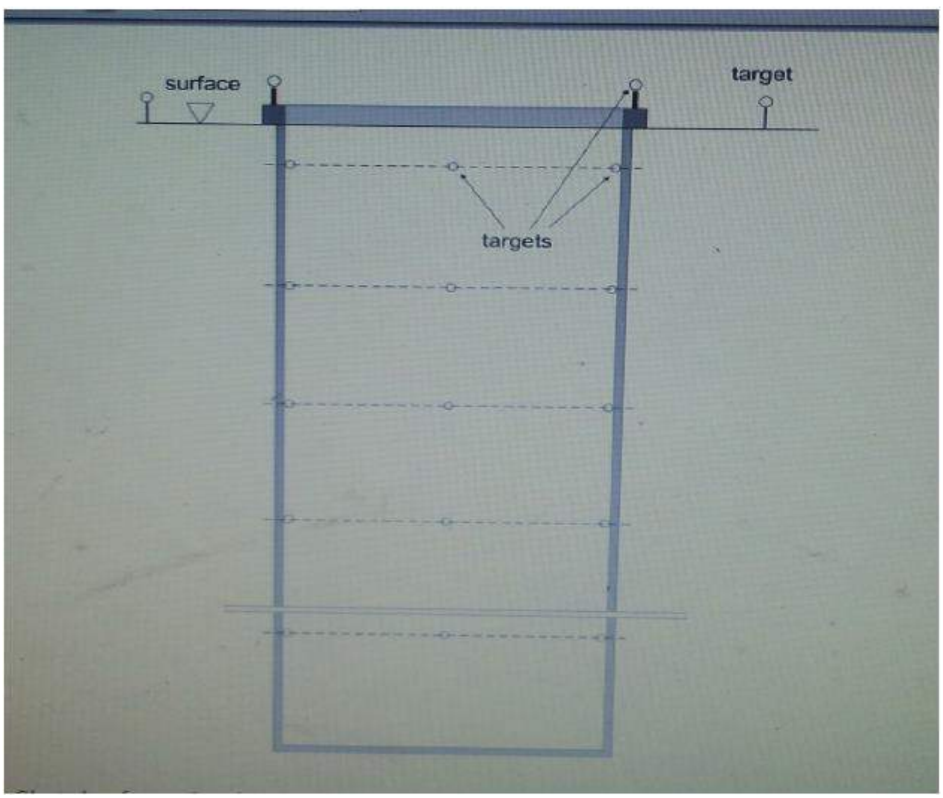


Figure 25. Sketch of monitoring layout for a circular shaft. [28]

C. Surface

Installation of the instruments and the zero readings of surface measurements shall be done well ahead of the;

- Excavation and / or
- Ground water table lowering

It is advisable to take at least two readings, which are not influenced by construction activities. Additional measurements may be required at request of the authorities.

D. <u>Surface target installation</u>

The following figures depict various types of possible monitoring target installation,suitable for application on native ground surfaces, traffic ways, objects and railways

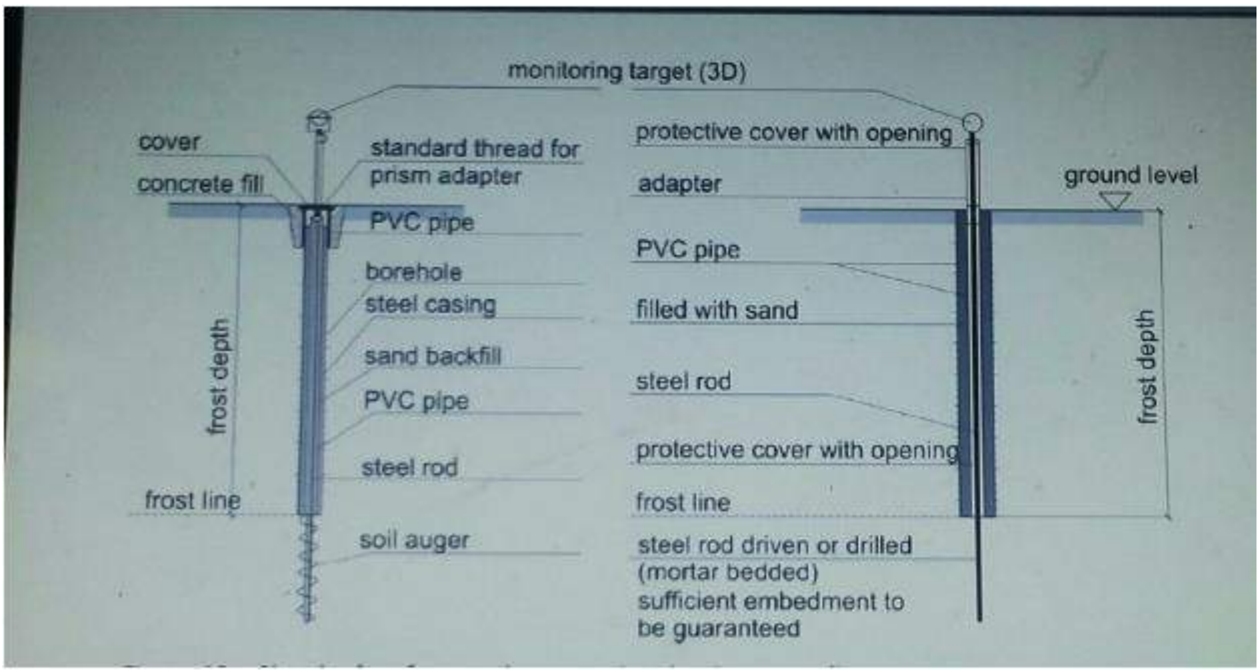


Figure 26. Sketch of surface settlement points (native ground) [28]

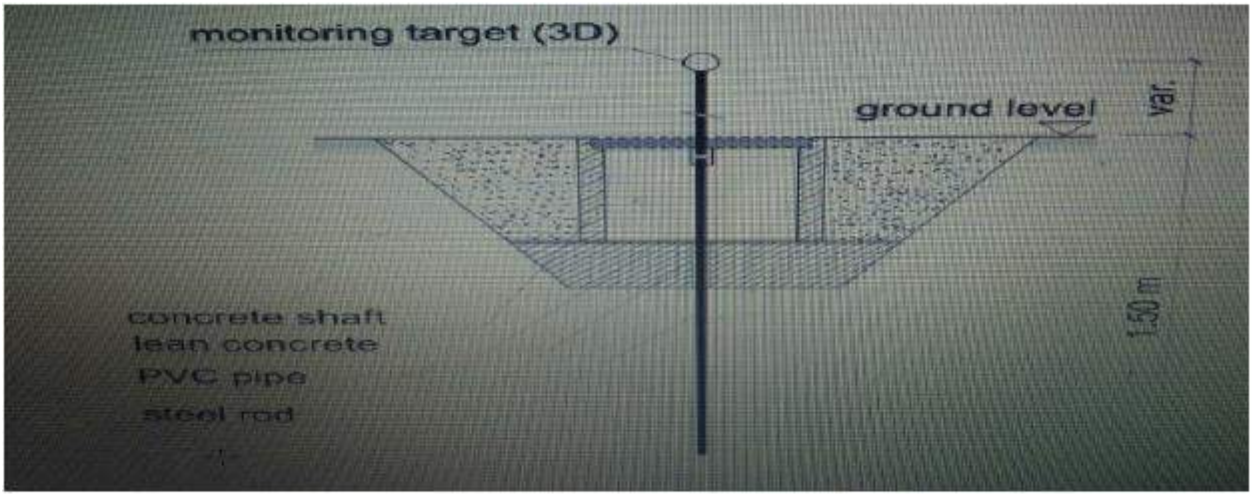


Figure 27. Sketch of a surface monitoring target in public areas [29]

5.5.2 Face displacement monitoring

The face monitoring system requires an automated measurement data acquisition systemwhich is conducting continuous tachymeter measurements without targets. Themeasurements taken on temporary obstacles should be automatically filtered (Figure 19).The maximum range is limited by the reflector-less distance measurement (approx. 100m) and the accuracy of the results is ± 1-3 mm, depending on the distance and onatmospheric conditions in the tunnel.

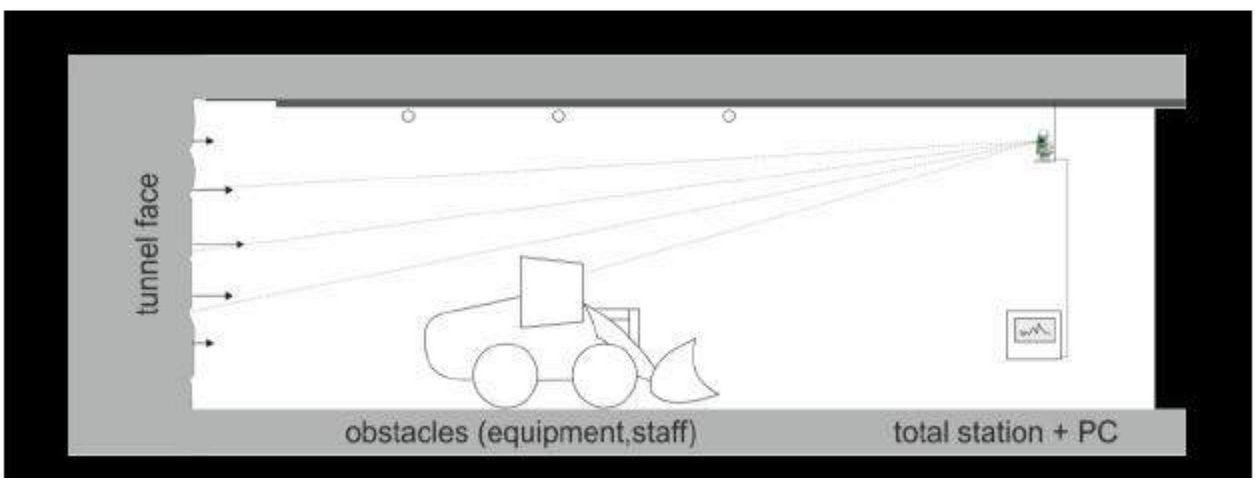


Figure 28. Sketch of the face monitoring system, modified after [29]

5.5.3 Levelling

For precise levelling, an electronic level with automatic recording unit in combination with an invert staff and marked points is used. The installation procedures presented in section(D) can be used for levelling as well. Required accuracy for a double levelling procedure (standard deviation) per 1 km: +/- 1 mm

5.5.4 Extensometers

Extensometers are used for determining ground movements by measuring the shortening or elongation (relative displacement) between two points. They allow an assessment of the development of strains and stabilization of movements in the surrounding ground.

5.5.4.1 Types of extensometers

There are various types of extensometers; however, all of them yield the same measurement data. Typical extensometer types are:

- Single or multi point rod extensometers, consisting of mechanically anchored measurement rod(s) and a protective pipe, allowing unhindered relative movement (Figure 20)
- Probe extensometers, consisting of a probe guide tube, points connected to the ground and a moveable reading element the measurement principles vary (mechanical, non-mechanical, etc.). Typical examples of such instruments are sliding micrometer probes and magneto strictive probes. They allow measuring at several depths along one probe.

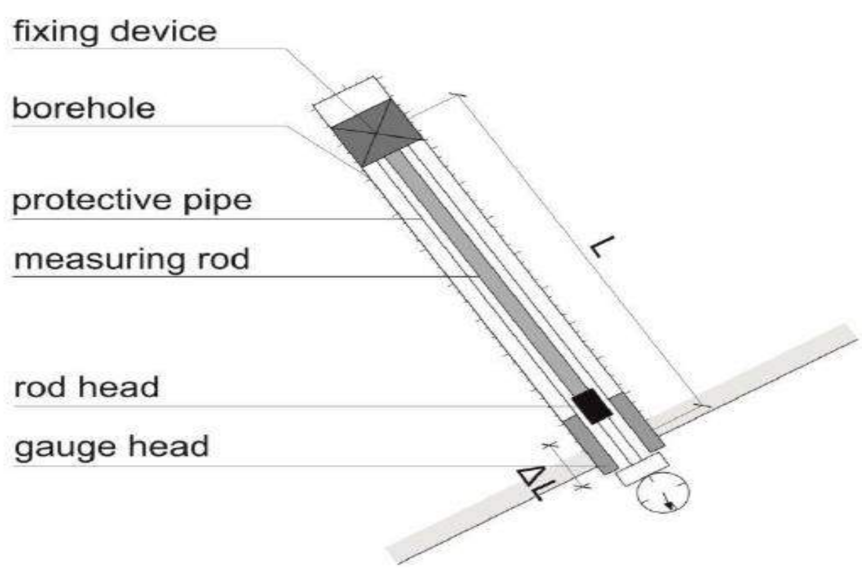


Figure 29. Schematic layout and principle of a single point rod extensometer [30]

5.5 Anchor load cells

Anchor load cells measure the loads in anchors and are installed at the anchor head (Figure 21).

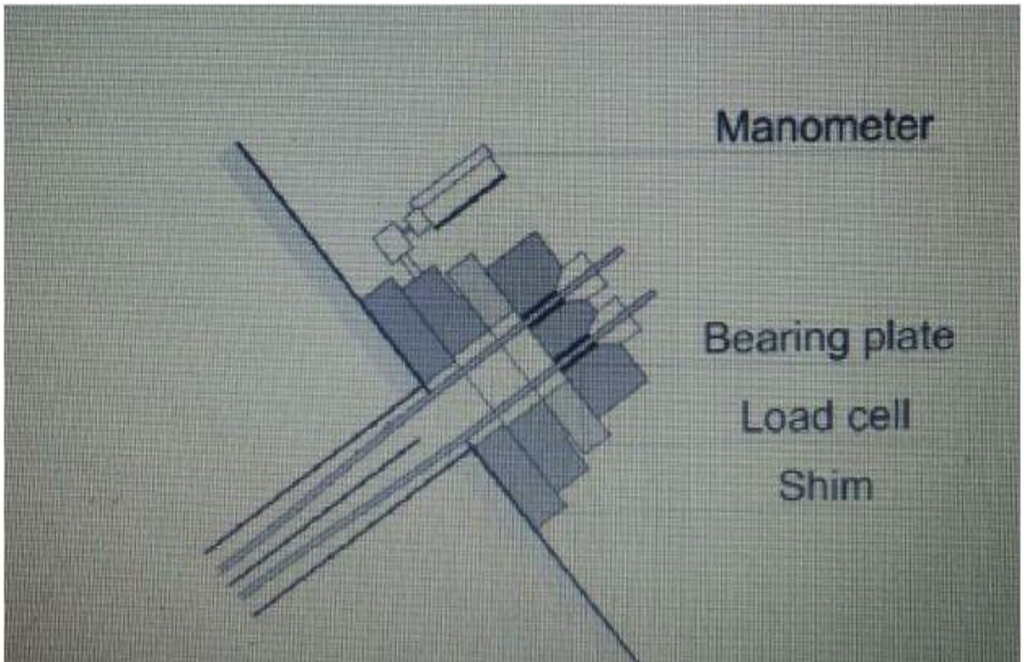


Figure 30. Schematic view of a load cell for a prestressed anchor [30]

5.5.5.1Types of load cells

Measurement of the loads is either done by a cell filled with oil, or cells equipped with strain gauges. Values can be either read out by manometer or remotely.

5.5.6Tilt meters

Tilt meters are used for measuring very small changes of the inclination, either on the ground or in structures. Such changes may be caused by excavation, tunneling, dewatering, or loading of the structure.

5.5.6.2 Types of tilt meters

There are different types of tilt meters:

- Electro level tilt meter
- Optoelectronic tilt meters

5.5.7 Hydrostatic leveling system

Water levels are used for the continuous monitoring of changes in height of buildings and other technical constructions (Figure 31).

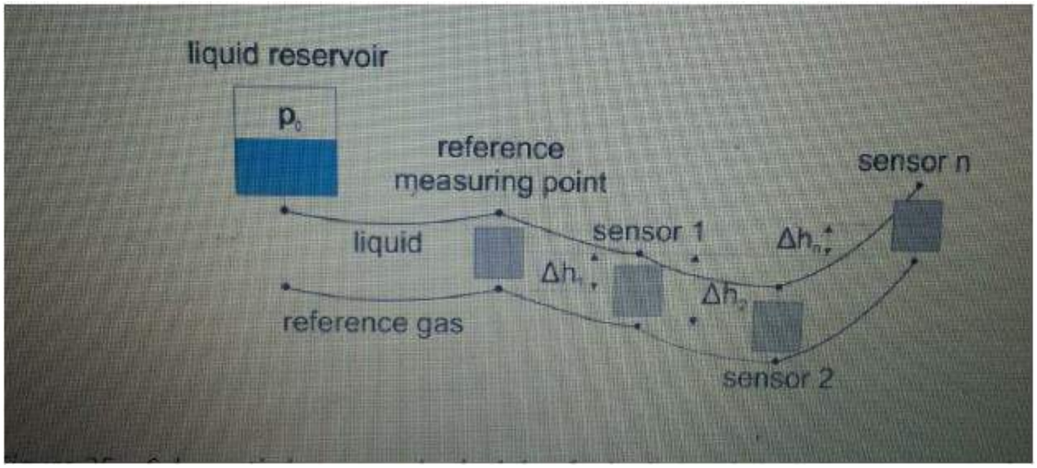


Figure 31. Schematic layout and principle of a hydrostatic levelling system [31]

5.5.7.2 Types of Hose Water Levels

The following types of water levels are used:

- Hydrostatic levelling system using manometers (Figure 22)
- Electrical water level systems
- Mechanical water levels

5.5.8 Borehole Inclinometers

Inclinometers are used for determining ground movements by measuring angular deflections (Figure32). They are commonly installed from the surface in the vertical andhorizontal direction. Also the installation from the tunnel in +/- horizontal direction ispossible. Combination with extensometers (sliding micrometer) is possible.

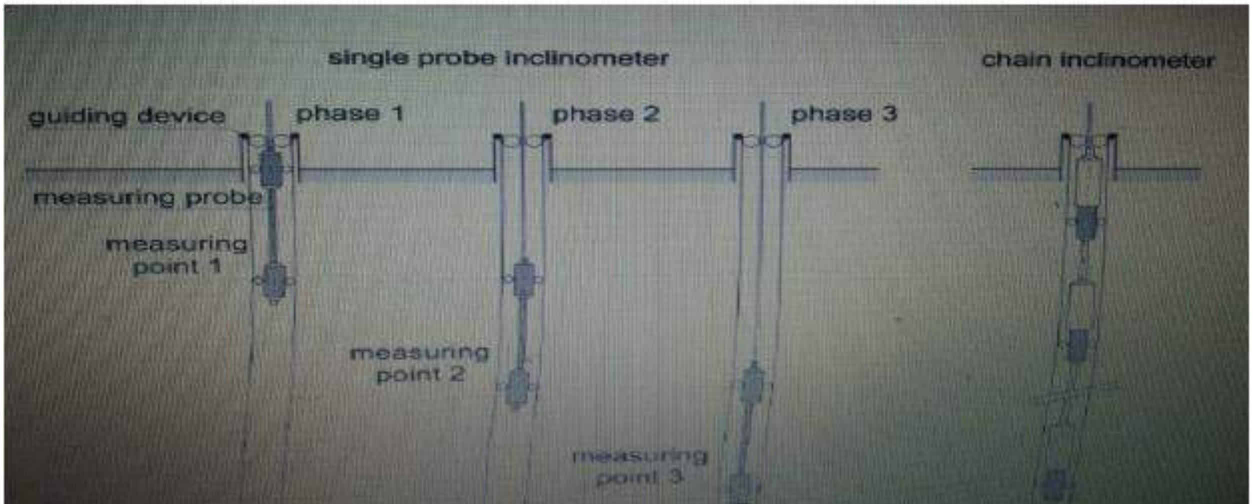


Figure 32. Principle of single probe and chain borehole inclinometers. [31]

5.5.8.1 Types of Inclinometers

There are two main types of inclinometers:

- Single probe inclinometers
- In-place inclinometers (chain inclinometer)

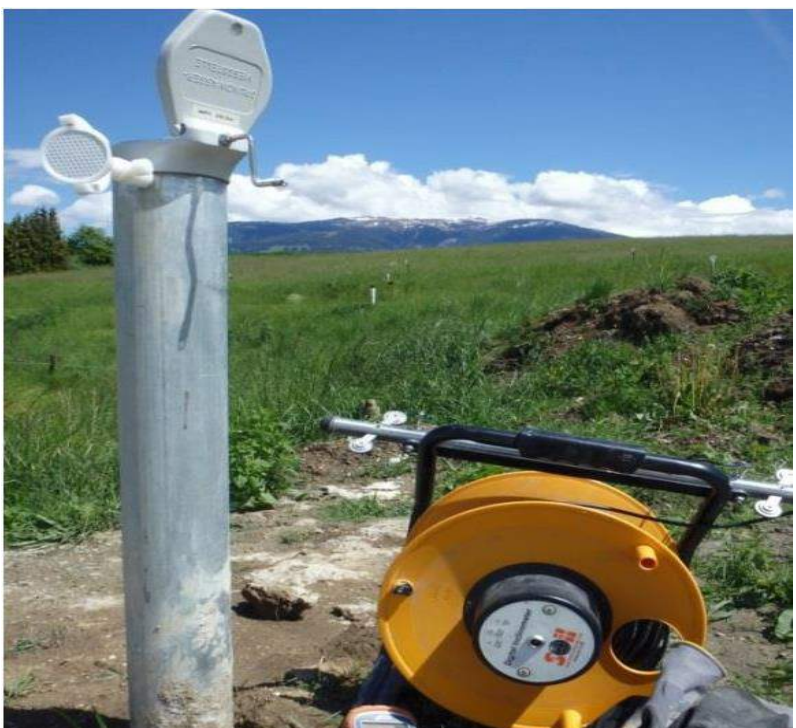


Figure 33. Head of borehole inclinometer with 3D displacement monitoring target. (Photo: Weissnar)

5.5.9 Water Level Gauges and Piezometers

Water level gauges are used to measure the water level in a borehole, by using a light plummet or sounding device. Piezometers are used for determining ground water pressure within a borehole (Figure 25).

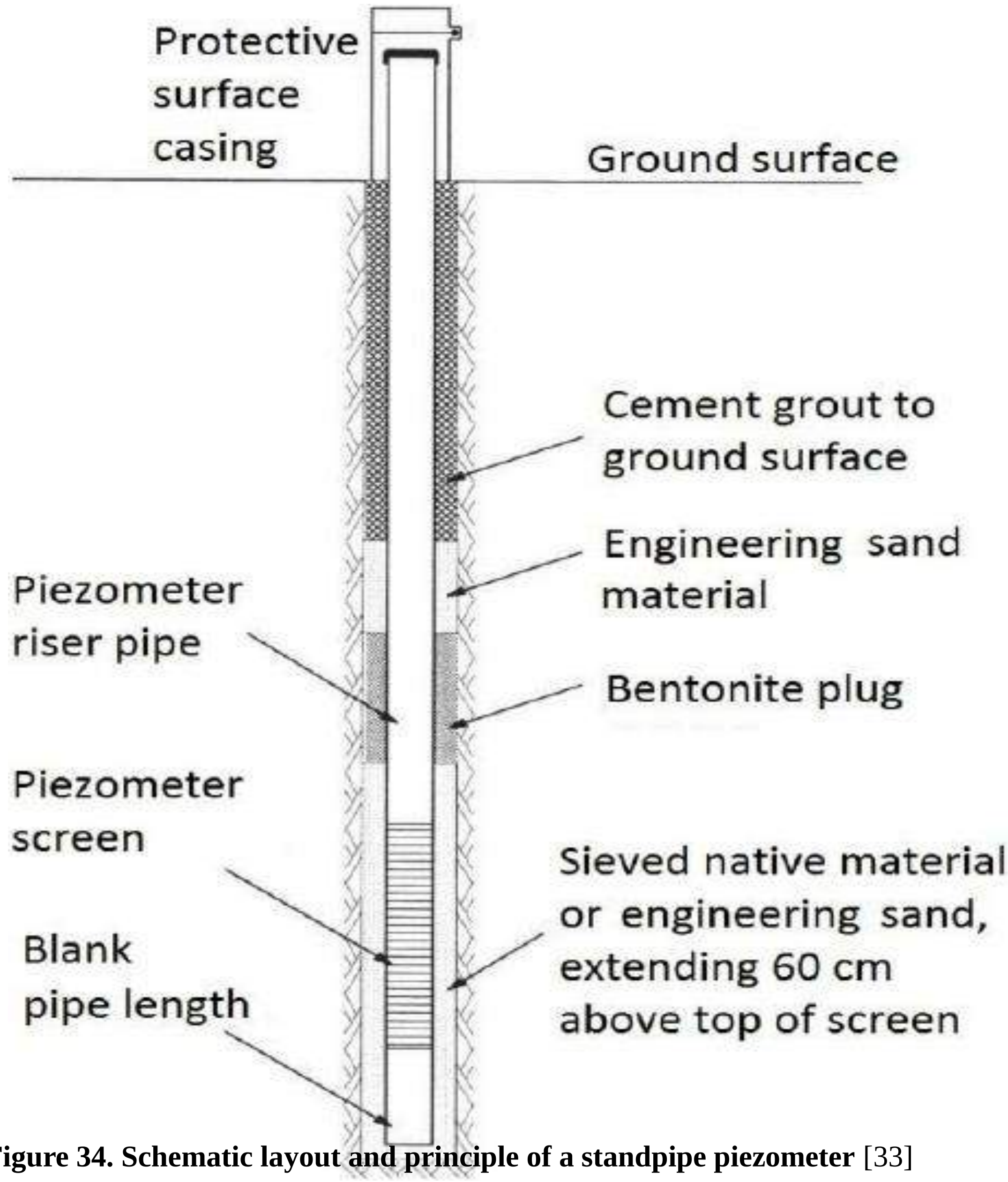


Figure 34. Schematic layout and principle of a standpipe piezometer [33]

3.5.9.1 Types of piezometers

The following types are commonly used:

- ✓ Single or multiple standpipe piezometer (Casagrande type)
- ✓ Standpipe

- ✓ Vibrating wire

5.5.10 Strain gauges

The following strain gauges are commonly used:

- ✓ Strain meters
- ✓ Vibrating wires (for cured concrete structures e.g. inner lining)
- ✓ Fibre optical sensors

5.5.10.1 Strain meters

Strain meters are used for measuring strains in shotcrete or inner linings (see Figure 36).

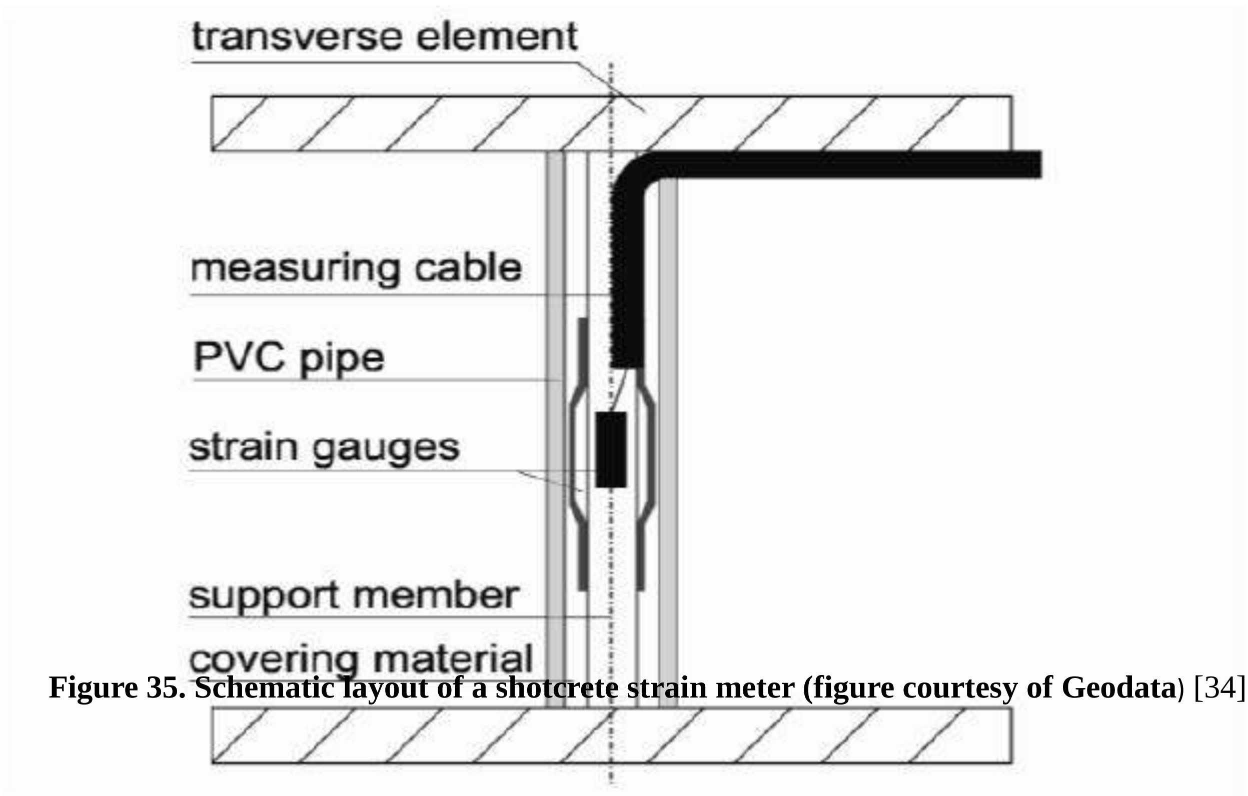


Figure 35. Schematic layout of a shotcrete strain meter (figure courtesy of Geodata) [34]

5.5.10.2 Vibrating wire sensors

Vibrating wire sensors are used to measure length changes along a measuring line. The layout is the same as for strain meters.

5.5.10.3 Fiber optic sensors

Fibre optical sensors are used to measure length changes along a measuring line.

Figure 35. Fiber optic sensor installed [35]

Figure 36. Pair wise installed strain gauges in the shotcrete lining of a shaft and arrangement of the wiring [35]

Figure 37. Vibrating wire sensors attached to reinforcement [36]

5.5.11 Invert probe

The invert probe is used for verifying the integrity of the invert which usually is covered with roadway material and therefore cannot be inspected visually. The device consists of a cylindrical probe which is connected to a stranded wire and inserted in a PVC-hose.

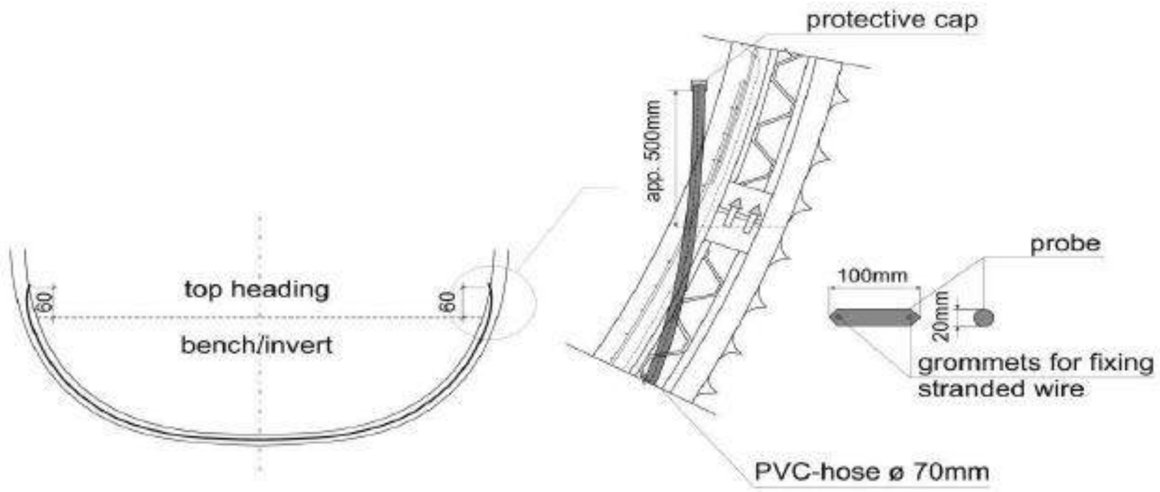


Figure 38. Typical invert probe (details and arrangement in cross section) [36]

5.5.12 Measurements of discontinuity location and orientation

An essential task to evaluate discontinuity-controlled failure modes is the determination of the spatial orientation and location of discontinuities. Conventional rock mass characterization requires physical access to the rock surface for applying a compass- clinometer device.

5.5.12.1 Manual structure mapping

The acquisition of spatial orientation of discontinuities is done with the help of a compass clinometer device (Figure 31). The device is located on a discontinuity (Figure 32), and dip and dip direction read from the instrument. The result usually is plotted in a face sketch.

Figure 39. Compass-clinometer (Clar system) [37]

Accuracy of measurement: reading accuracy is approximately 1° for the dip direction, and approximately 2°-3° for the dip angle. Deviations in the orientation measurement can be caused by magnetic field or steel elements installed in the tunnel. Shortcomings of this measuring technique are:

- ✓ Readings taken manually in potentially hazardous area
- ✓ Readings only possible in accessible areas
- ✓ Readings not reproducible after excavation of next round
- ✓ Location of features generally only estimated

Figure 40. Measuring of discontinuity orientation with compass-clinometer device at a tunnel face (Photo: GeoteamKoralmtunnel, lot 3) [37]

5.5.12.2 Digital ground mapping

Two technologies are mainly used for contact free (remote) measuring of discontinuities and other geological features: Digital photogrammetry and LiDAR (light detection and ranging).
Major advantages of these remote techniques include:

- The ability to quickly acquire large portions of rock masses including inaccessible areas
- The possibility to zoom in and out of an outcrop, which leads to a better understanding of large features
- Permanent documentation of the rock face condition and excavation stages for reporting and contractual-legal issues

The general work-flow when applying these techniques includes collecting data by taking photographs or scanning the region of interest from distant locations and processing the acquired data to a 3D surface description (3D image) with a photo overlay (Figure 33).

A. Accuracy

In the context of remotely acquiring surfaces for geologic mapping, accuracy shall be seen twofold:

a. the accuracy of single surface point measurements (positional accuracy), and
b. the accuracy of the entire surface description (shape accuracy) which is strongly related to the term resolution or point density.

Both mentioned technologies have the capability to scan/acquire surfaces at different resolutions and different positional accuracies. The higher the resolution, the smaller are the details that can be mapped from the resulting 3D data. It is therefore reasonable to consider the requirements of a project first and then choose the measurement/documentation tool and its resolution accordingly. In the case of tunnel face mapping, it is generally sufficient to map geologic structures in the sub-centimeter range. Assuming the sectional area of a tunnel being about 60 m2 and it is photographed with an 18 MPixel camera, a resolution of 3-5 mm/pixel and a positional accuracy error in the same range can be expected. With LiDAR similar resolution can be achieved.

Figure 41. 3D image of a tunnel face and lining (Photo: 3GSM) [38]

Table 2. Installation requirements for different type of instrument [39]

Item	Instrument Type	Installation requirements
1	Extensometers	□ In case of rod extensometers, the head of the instrument shall be protected by a cap. A reliable bond between the anchors and the ground shall be ensured □ The extensometer head should be paired with a 3D displacement monitoring target. In case of surface installation, levelling of the extensometer head shall be conducted concurrently □ The position of the extensometer head should be determined immediately after installation, and parallel to the zero measurement of the extensometer □ The spatial orientation of the extensometer shall be Measured
2	Anchor load cells	□ The surfaces shall be planar, and the read out device accessible.
3	Tilt meters	□ The position of the tilt meter should be determined immediately after installation and parallel to the zero measurement of the tilt meter
4	Hydrostatic levelling system	□ For temperatures under 1°C a special non-freezable liquid shall be used □ The sensor positions should be paired with a 3D displacement monitoring target □ The zero reading of the monitoring targets should be done immediately after installation and parallel to the zero measurement of the water level

5	Borehole Inclinometers	➢ The inclinometer head should be paired with a 3D displacement monitoring target ➢ The position of the inclinometer head should be determined immediately after the installation and parallel to the zero measurement of the inclinometer ➢ The casing slots should be oriented such that they are parallel and perpendicular to the orientation of any tunnel excavation or wall to be monitored

6	Piezometers	➢ The position of the piezometer head should be determined immediately after installation ➢ The depth of the probe within the borehole should be Recorded
7	Fiber optic Sensors	➢ The position of the sensors should be determined and recorded ➢ The sensors should be fully embedded in the lining/ground ➢ Wires should be protected against damage
8	Invert probe	➢ The device shall be installed together with the shotcrete lining ➢ Both ends of the hose shall be accessible, protected against damage and covered with a cap

Table 3. Accuracy requirements for different type of instrument [40]

Item	Instrument type	Accuracy
		per 10 m measuring length, Measuring range: vertical or horizontal: +/- 30° (10° for in-place inclinometers)
1	Extensometers	✓ Common resolution of reading instrument: 0.01 mm
2	Anchor load cells	✓ Accuracy of hydraulic systems is +/- 1 % of the capacity of the load cell. Accuracy for electrical load cells are in the range of +/- 0.5 %. Error due to temperature change of 20° for both systems is in the range of 1.2 %.
3	Tilt meters	✓ The total system accuracy of tilt meters is between 0.005 m rad and 0.050 m rad, depending on the measuring range.
4 5	Hydrostatic levelling system Hose water levels	✓ The total system accuracy for hydrostatic water levelling systems are 0.3 mm for the measured height differences. The accuracy of hose water levels varies between 1 and 2 mm. The measuring range for all systems is about 200 mm. The total system accuracy for hydrostatic water levelling systems is 0.3 mm for the measured height differences. The accuracy of hose water levels varies between 1 and 2 mm. The measuring range for all systems are about 200 mm.
6	Borehole Inclinometer	✓ Common resolution of reading instrument: ± 0,02 mm per 500 mm gauge length, The system accuracy shall be better than 1 mm

7	Piezometers	✓ Common resolution of reading instrument: ± 0.025 % of full scale, The system accuracy shall be better than ± 1 % of the full scale, The operating range of the system shall not exceed the expected maximum pressure by more than 50 %
8	Strain meters	✓ Resolution of reading instrument: ± 0.01 % of full scale, The system accuracy shall be better than ± 1 % of full scale
9	Vibrating wire sensors	✓ Resolution of reading instrument: ± 0.01 % of full scale, The system accuracy shall be better than ± 1 % of full scale
10	Fiber optic sensors	✓ For strain measurements in structural elements, like concrete, steel or shotcrete, the following, requirements should be met: System accuracy: 1 μm/m, For measurements of ground movements, the

5.6 Monitoring of displacement and stress around a tunnel under construction.

Deformation control during tunneling excavation is becoming over the last decades a basic requirement for both safety purposes and for faster production. The monitoring of deformation may allow to verify if the deformation at the front or in the tunnel are in line with the expected ones. Hence, it is possible to understand if we are excavating under the predicted design condition and, if not, to adopt Effective countermeasures. Italy, together with the Japan has the longest tunnel network in the world, including different types of tunnels and, especially, tunnels excavated in different materials, both hard and Soft rock and soil. Over the last 30 years a new tunneling excavation method has become popular, thus substituting the common NATM method. This method, named ADECO-RS (an Italian acronym for "Analysis of controlled Deformation in rock and soil") (Lunardi, 2008; Tonon, 2010), considers the deformation as the core of the tunneling activity and, therefore, the control of deformations during the excavation is a key requirement for each planning decision, before and during the excavation phase. Together with the ADECO-RS method, several excavation and stabilization solutions have been developed as well as new approaches for the monitoring of deformations. Hence, deformation monitoring is now at the base of a tunneling project. Furthermore, in case of tunnels crossing an unstable slope several well established traditional monitoring techniques (inclinometers, extensometers, extensor-inclinometers, topographical surveys etc.) are used. Following the great attention given to the observational method, new solutions for the monitoring of slope deformation have been developed in the last years. New branches of deformation monitoring have been created like remote monitoring, i.e. monitoring by lasers, radar etc (Mazzanti, 2012), thus making the monitoring a large and complex science. These techniques are also offering new opportunities such as the deformation monitoring as a tool for investigation purposes (Mazzanti, this volume). Deformation monitoring surveys were conducted and measurements of movements were carried out throughout the construction cycle of the wall and beyond. In order to determine design parameters for the soil strata embedded in a complex geological sequence, the soil-wall interaction was back analyzed using the finite element method [41]. Deformation Monitoring Techniques in this study, currently existing diversified deformation monitoring techniques is being addressed. First, Tunnel Wall deformation is usually measured with tape extensometers, geodetic surveying (total stations) and laser scanners (profilometers). Laser scanners (or tunnel profilometers) are recent development in measuring the geometry of tunnel walls in cross-section. A typical system shown in Figure 1 consists of two closed-circuit digital (CCD) cameras mounted on a portable frame [25]. The position of the camera frame is automatically determined by a total station with automatic target recognition placed up to a maximum distance of 100m. For this purpose, three reflector targets are permanently mounted on the frame. Digital images are automatically stored in a computer and can be processed to provide the 3D coordinates of the surveyed tunnel wall surface with an accuracy of F5 mm for each coordinate. Although this level of accuracy is low compared to routine geodetic surveying, the advantage of recording a very large number of points on the tunnel

wall can outweigh low accuracy for many applications. Second, deformation measurements at ground surface, structures and utilities are usually performed with surveying instruments (precision levelling for vertical displacement and total stations for 3D geodetic facade monitoring), or with geotechnical [42].

CHAPTER SIX

6. Monitoring Scheme and Recommendation

Monitoring plays an important role in every stage of the construction of a tunnel:

- **At the design:**
  Stage involving an exploration tunnel for site evaluation or the doubling of an existing tunnel.
- **During construction:**
  -To accurately evaluate the impact of geological conditions, the effect of the tunnel on nearby structures and construction methods to be used. Design hypotheses can be confirmed, the needs of the support structures defined and the optimum moment for instrument installation in accordance with convergence- confinement (NATM) methods can be determined.
- **Once the tunnel is in service:** To enable long-term monitoring thus ensuring the safety of the tunnel over its life span. Instrumentation is used to accurately quantify certain parameters of structural behavior and to monitor their rate of change. It is possible to observe movement stabilization, or, in the case of acceleration, to deduce the possibility of failure. The comparison of measured values with design values enables the monitoring of tunnel stability and the possibility of implementing corrective measures at the appropriate moment. Certain projects, such as tunnels in soft grounds, carried out in urban areas, would be practically impossible to achieve without instrumentation and automated data acquisition systems.

6.1 Selection Criteria for Instrumentation

The chosen type of instrumentation shall guarantee the following: -

- Feasible installation procedure
- Durability over the monitoring period
- Protection against damage during construction
- Simple measurement handling (data acquisition and transmission)
- Accuracy as required

6.2 Types of Measurement

The types of measurement and instrument location must be adapted to the existing geological and environmental conditions as well as construction methods. Therefore, there are certain general guidelines to follow for instrumentation selection, based on tunnel construction.

6.2.1 Cut and Fill Tunnel

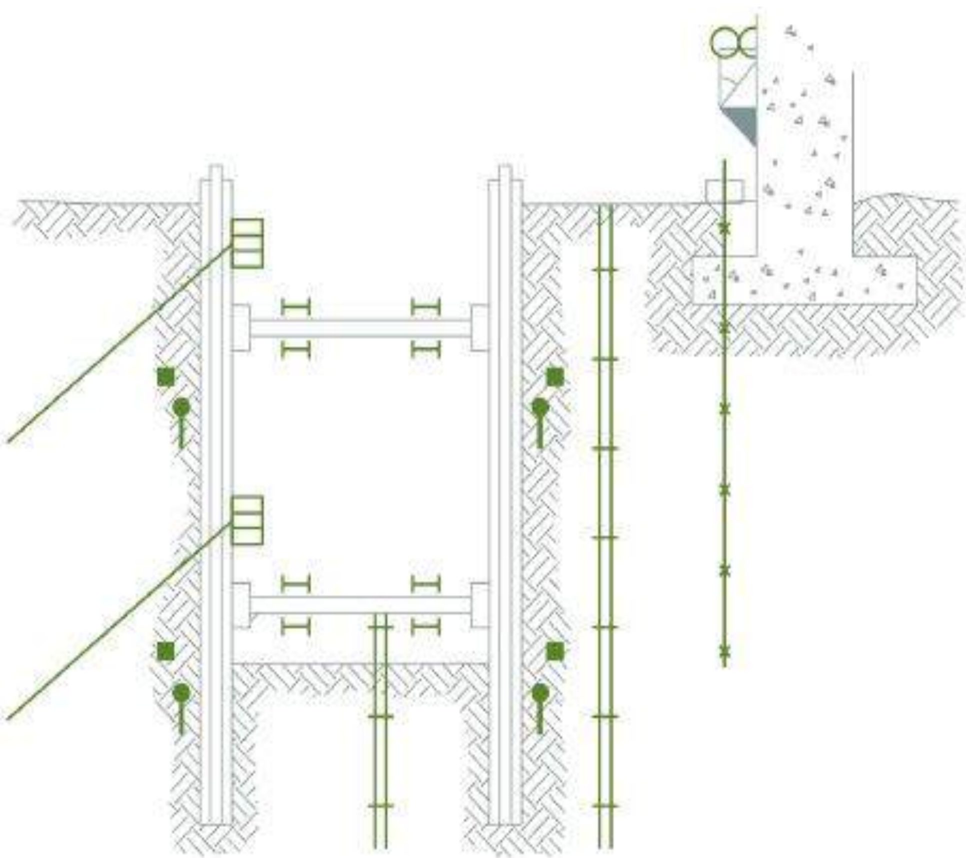
Objectives: -

- Ensure stability in the retaining walls
- Monitor the integrity of adjacent structures

Measured Parameters: -

- Stress in struts
- Load in anchors
- Deformation of adjacent ground and buildings

Figure 42. Cut and cover tunnel with instrumentation [42]

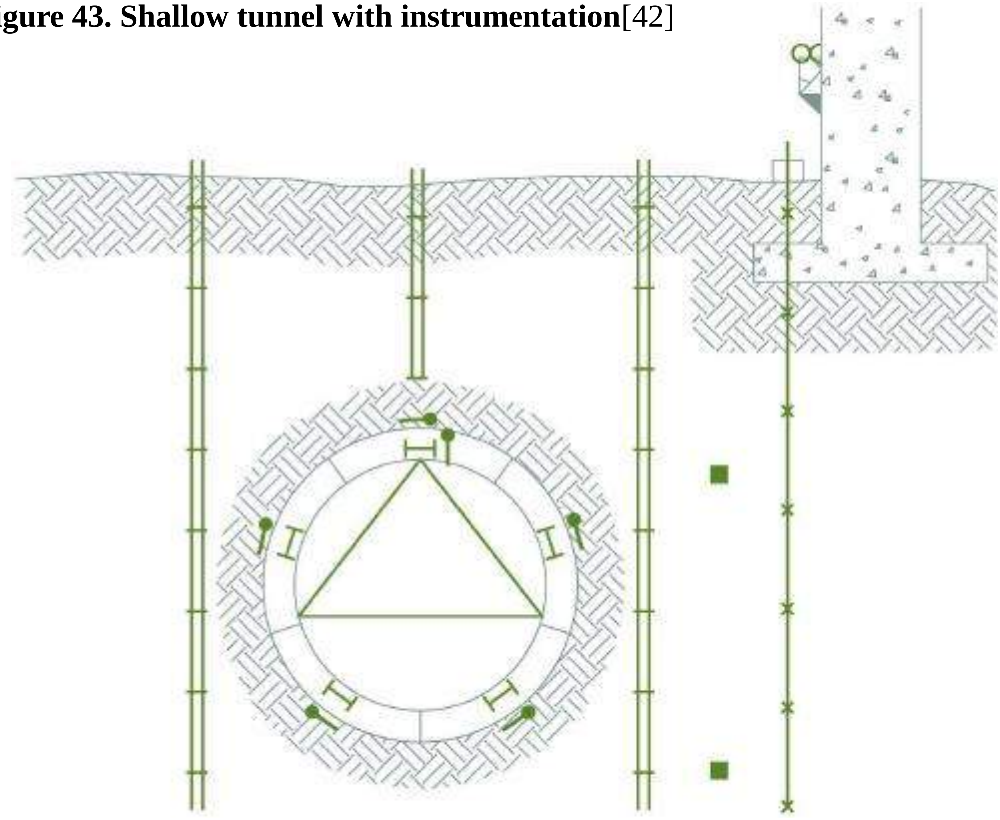


6.2.2 Shallow Tunnel in Unconsolidated Ground

Objectives: -

- Monitor the integrity of adjacent structures
- Ensure stability in lining segments
- Measured Parameters: -
- Stress in lining segments
- Load on tunnel linings
- Convergence
- Ground settlement
- Deformation of adjacent buildings

Figure 43. Shallow tunnel with instrumentation[42]

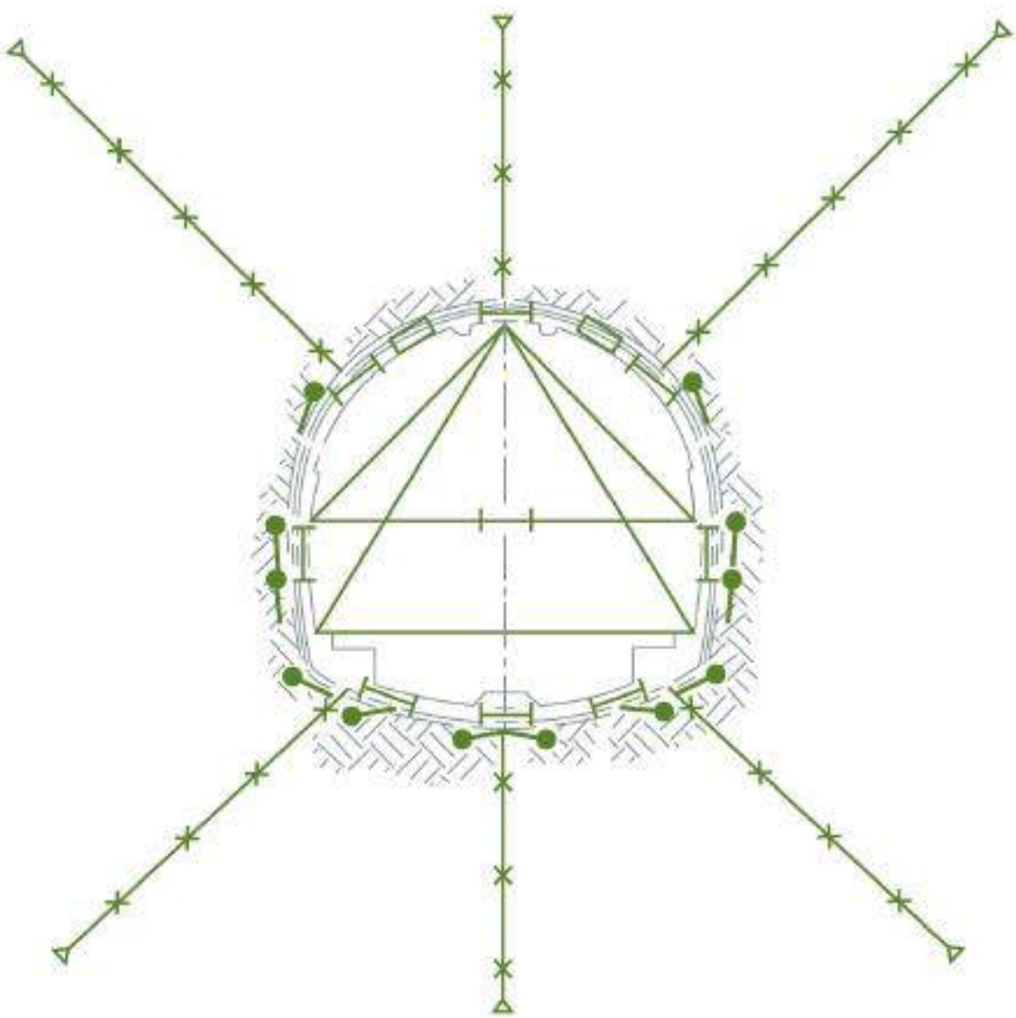


6.2.3 Deep Tunnel in Rock

Objectives: -

- Ensure the stability of the excavations
- Monitor the integrity of the tunnel lining Measured Parameters
- Convergence
- Deformation of the rock mass around the excavation
- Stress in the lining

Figure 44. Deep tunnel in rock with instrumentation [42]



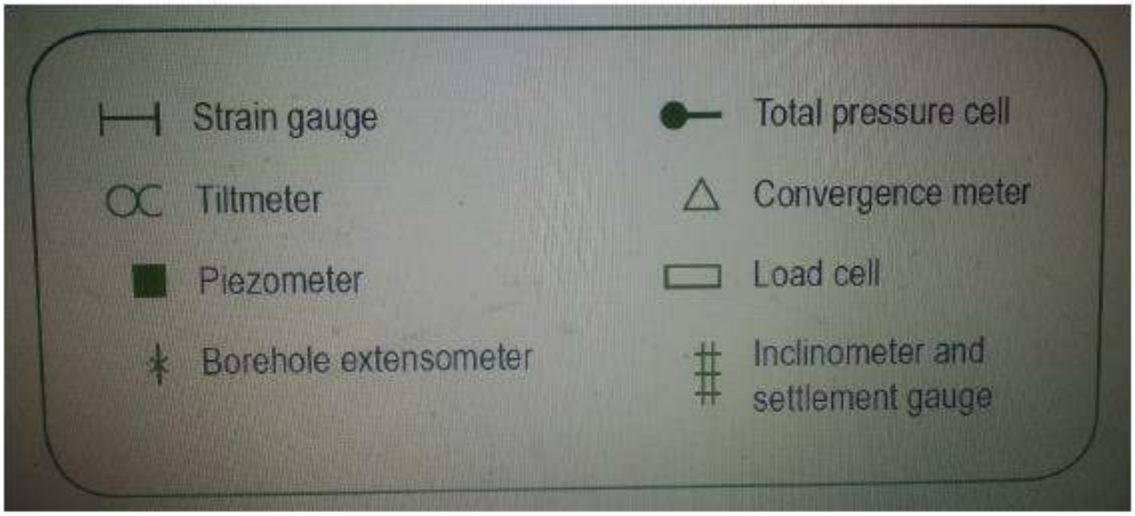


Figure 45. Legend for Fig 34, 35.36

6.3 Monitoring concept and identification of key observation variables

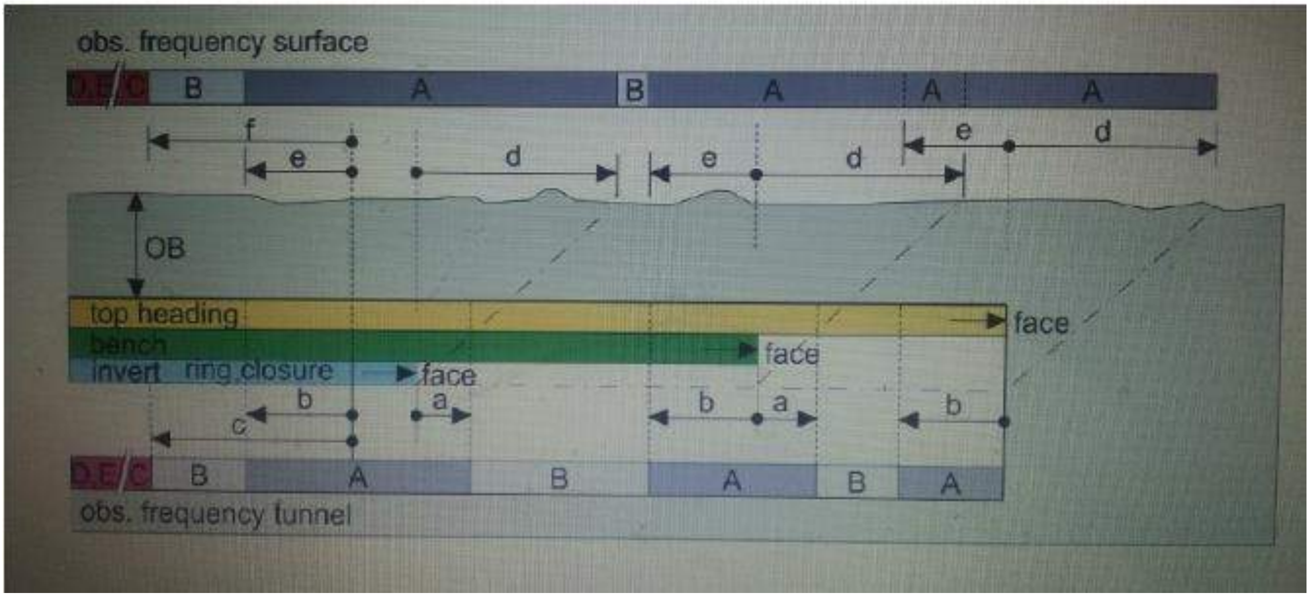
The arrangement and layout of the various monitoring installations should be chosen under consideration of the specific project requirements and boundary conditions. The type, number, or spacing of the monitoring installations should consider the construction layout and sequence, the quality of the encountered ground and the surface situation in terms of sensitive structures (e.g. buildings, infrastructural utilities, etc.).

6.4 Reading frequency

6.4.1 Tunnels

In general, readings close to excavation activities are taken daily; the frequency is reduced with distance to the face and decreasing rates of displacements. Shorter monitoring intervals can be required due to project specific requirements. A possible concept showing minimum reading frequencies and monitoring ranges for surface and underground monitoring for a top heading-bench-invert sequence is illustrated in Figure 45. [44]

figure 45 observation for frequency surface. [44]



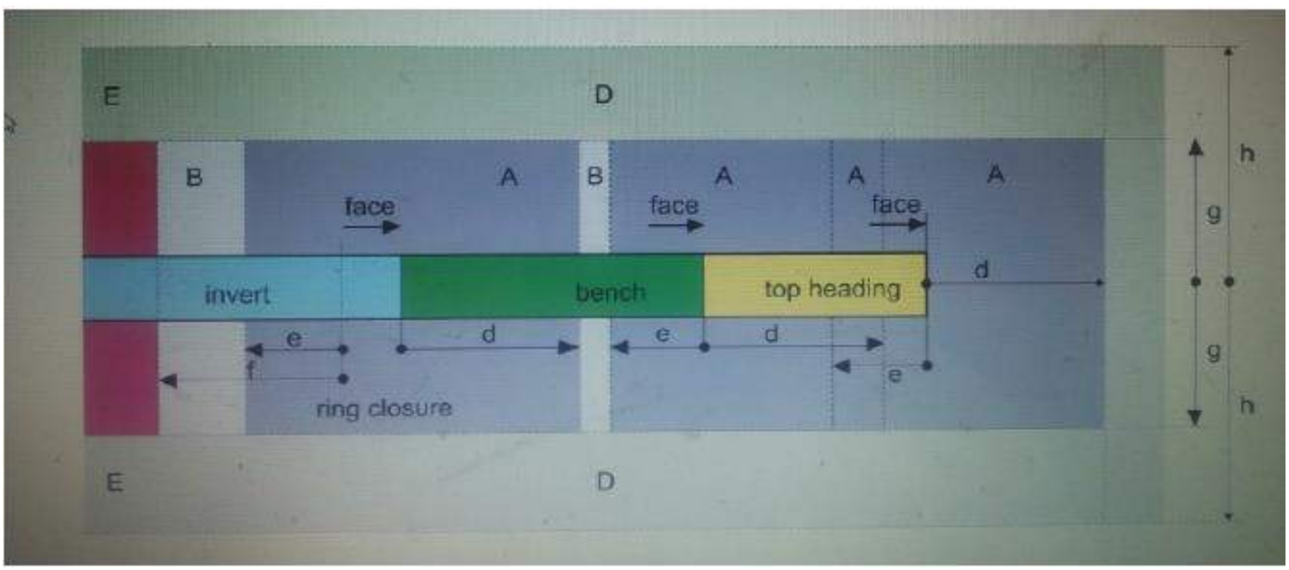


Figure 46. Sketch of areas with different reading frequencies for a shallow tunnel; longitudinal section (top), and plan view on surface level (bottom); OB: overburden

It is distinguished between zones, directly influenced by tunnel excavation, and zones which are outside of this range but within a fixed observation corridor (i.e. monitoring points on buildings). The values are minimum requirements, which are increased by the parameters “Xi” (Variable lengths) and “n” (number of monitoring sections (MS)) to consider influences of lowering of the ground water, building location, special ground conditions, etc. The parameters “Xi” have to be determined on a project specific basis. For other construction sequences, like side wall galleries, parallel headings, etc. the above recommendations also apply.

Table 4. Spatial differentiation of monitoring ranges related to construction phases

	Distances	Monitoring range
Tunnel (Ø: tunnel diameter)	A	2Ø+X1 and min. (2+n)Ms
	B	3Ø+X2 and min. (3+n)Ms
	C	5Ø+X3 and min. (5+n)Ms
Surface	D	Ø+OB+X4
	E	3Ø+X5
	F	5Ø+X6
	G	1,5Ø+OB+X7
	H	3Ø+2OB+X8

Table 5. Typical minimum reading frequencies in the respective monitoring ranges from the geotechnical point of view

Range	Min. observation frequency
A	1 per day
B	2 per week
C	1 per week
D	1 per month
E	as required

6.4.2 Long term monitoring

In ground with pronounced time dependent behavior (swelling ground, ground with tendency for creeping, consolidation, etc.) the system should be monitored also after completion of construction works. It may be required to continue monitoring during operation.

6.4.3 Typical distances between monitoring sections

Monitoring sections in tunnels and shafts are typically situated at distances of 5 - 20 m depending on the boundary conditions and requirements

CHAPTER SEVEN

CONCLUSION

The use of instrumentation is an essential part of tunnel construction today. Geotechnical instrumentation is different from any other type of instruments in that it needs a comprehensive and complete interaction between the designers, the user and the instrument supplier. Proper installation of geotechnical instruments is as important as the quality of instrument itself since once embedded, the instrument cannot be taken out. If an instrument has failed after installation, it cannot be replaced. Therefore, we should be used instrumentation and monitoring of tunnels for different ground condition based on availabilities and use of monitoring as described in this paper for proper monitoring to avoid any default in tunnel construction and operation. Otherwise no benefit can be obtained from these instruments unless the instruments are installed properly, the data monitored regularly and made available to the user readily. And last but not the least, the data must be interpreted and made use of by the user at all times.

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